VOLUME 86 NO. PO4
AUGUST 1960
PART 1

JOURNAL of the

Power

Division

PROCEEDINGS OF THE

AMERICAN SOCIETY

OF CIVIL ENGINEERS

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This Journal is published bi-monthly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class postage paid at Ann Arbor, Michigan.

The index for 1959 was published as ASCE Publication 1960-10 (list price $2.00); indexes for previous years are also available.

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Journal of the

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Proceedings of the American Society of Civil Engineers

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CONTENTS

August, 1960

Papers

Page

Digital Computers for Trial-Load Analysis of Arch Dams
by L. R. Scrivner 1

Design of Arch Dams by Trial Load Method of Analysis
by Merlin D. Copen. 13

Design of Karadj Hydroelectric Project
by Richard D. Harza and Robert F. Edbrooke 31

DISCUSSION

Bureau of Reclamation Practices for Control of Cracking in
Arch Dams, by Charles L. Townsend. (August, 1959.
Prior discussion: February, 1960. Discussion closed.)
by Charles L. Townsend 53

(over)

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Page

Errata 57

Journal of the
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DIGITAL COMPUTERS FOR TRIAL-LOAD ANALYSIS OF ARCH DAMS

By L. R. Scrivner[1]

SYNOPSIS

Stress analysis of concrete dams is another of the ever-growing list of engineering problems that can be expedited by the use of electronic computers. The use of the "trial-load method" has been restricted due to the time-consuming, laborious computations that were required. The programming of many of these operations for an electronic computer results in a saving of time and costs, making this method of analysis more effective in the design of arch dams.

INTRODUCTION

The trial-load method of analysis has been used extensively, since 1930, for designing and checking the designs of arch dams in the United States and many foreign countries.[2]

In 1957, the Analysis Unit of the Concrete Dams Section of the United States Bureau of Reclamation, Dept. of Interior, (USBR) began the development of a series of electronic computer programs to reduce the time and cost required to complete a trial-load study. Programs have been developed utilizing the IBM 650 digital electronic data processing machine (system), which has a storage capacity of 2,000 "words." A word as referred to here is a number containing 10 decimal digits and a plus or minus sign. The machine used is a

Note.—Discussion open until January 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 4, August, 1960.

[1] Head, Stress Analysis Unit, Concrete Dams Section, Division of Design, Bureau of Reclamation, Denver, Colo.

[2] "Arch Dams: Review of Experience," by Robert E. Glover, Power Division, American Society of Civil Engineers, Paper 1217, 1956.

stored-program, modified single-address, numerical, decimal machine. The decimal digit representation of the input data is translated into bi-quinary representation for use and transmission within the computer. The use of this bi-quinary code makes possible the validity checks employed in the machine on the data as it is transmitted and transferred internally. Instructions and data are stored on a cobalt-nickel plated drum in 40 bands of 50 words each in the form of magnetized spots. Each digit is represented by a specific recognizable pattern of these spots.

Most of the programing was done using a modified form of an interpretive routine that was developed to handle floating decimal arithmetic including the computation of the elementary transcendental functions.[3] Because it primary purpose is to provide floating operations, it has been referred to as "Flops" and was used by Quintin B. Graves in the analysis of pipeline networks.[4] Input, output, and intermediate data are used and stored in a floating-decimal form. The 8 most significant digits occupy the left 8 digit positions of a data word. The right two digits of a data word indicate the position of the decimal point; for example +Pi would be written as:

+3141592750

The 50 indicates that the decimal follows the first digit and is referred to as the machine exponent of the number.

"Flops" operation codes have five parts. These parts are Sign, 0_1, A or 0_2, B, and C. The sign will be positive except when the instruction is to be modified by a "loop" instruction. The operation code may be a single digit 0_1, (1 through 9), or with 0_1 equal to zero, 0_2 will be a 3-digit operation code. The interpretive routine uses the storage locations with addresses 1000 through 1999. The A, B, and C addresses are, therefore, 3-digit addresses, 000 through 999, with a leading zero inferred. The "Flops" system provided exponential functions and tracing routines as operation codes, but because they were not required by this problem they were removed and the space occupied by them was made available to machine language programs by means of an availability table. The operation codes are summarized in Table 1. The asterisks therein indicate additions or changes from the original "Flops" codes. The changes in "Flops" codes and other machine language programing was done using the Symbolic Optimum Assembly Program "SOAP II."[5]

PROCEDURE

The electronic computer has, to date, been used primarily for the computation of forces and deflections in arch elements due to unit arch loads. This approach has been followed because of the following:

1. Approximately 70% of the man-hours, and therefore the cost of performing a trial-load study, is expended making these computations.
2. These computations are repetitive in nature, which is a factor favoring advantageous use of electronic computers.

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[3] International Business Machines, Technical Newsletter, No. 11, New York, N. Y.

[4] "Digital Computers for Pipeline Network Analysis," by Quintin B. Graves, Sanitary Division, American Society of Civil Engineers, Paper 1608, April, 1958.

[5] Symbolic Optimal Assembly Program (SOAP II) for the IBM 650 Data Processing System, Service Bureau Corporation.

Computations have been carried out following very closely the procedures for analyzing arch dams described in USBR publications.[6,7] Only minor changes were made as required to simplify the machine program. (A program is defined as a predetermined series or sequence of operations to be performed by electronic operation of the machine.) Following established procedures has made checking much easier and has permitted benefits to be obtained from incomplete programs. Table 2 lists some of the programs that are now in use

TABLE 1.—SUMMARY OF OPERATION (Op) CODES

0_1 Operations			0_2 Operations			
0	Operation 0_2		000	UNC STOP	204	TR SUBR
			200	COND STOP	205*	TR OUT (To 4 digit C)
1	ADD	A + B = C	201	TR SIGN	254*	TR (To 4 digit C)
2	SUB	A - B = C	202	TR EXP		
3	MPY	A · B = C	203	TR	300	SQRT
4	DIV	A/B = C	100	LOOP A	303	SINR
5	NGMPY	- A · B = C	010	LOOP B	304	COSR
6	TR A		001	LOOP C	305	ART R
7	TR B		110	LOOP AB	350	ABS
8	TR C		101	LOOP AC	353	SIN D
9	MOVE		011	LOOP BC	354	COS D
			111	LOOP ABC	355	ART D
			500	SET A	400	READ
			050	SET B	401	CONS
			005	SET C	410	PCH
			600	ADD A	411*	PCH 1 ALPH
			060	ADD B	412*	PCH 2 ALPH
			006	ADD C	450*	CLEAR
			700	SUB A	453	SWITCH
			070	SUB B	454	NO OP
			007	SUB C	800	COUNT

and gives input requirements. Programs presented herein will be referred to by the file number listed in Table 2.

COMPUTATION OF GEOMETRY

Program No. 182 was developed to compute the geometry of variable thickness dams which have a combination of straight lines and circular curves defining the plane of centers, as indicated in Fig. 1. The program as written is restricted to the general type of layout indicated, but can readily be modified for other types, as long as the elements of the plane of centers may be defined by mathematical equations that can be solved by usual computational methods.

Approximately 0.3 hr of machine time is required to compute dimensional data needed for a trial-load analysis of a typical arch dam having 5 to 6 arch

6 Trial-Load Method of Analyzing Arch Dams, Boulder Canyon Project Final Reports, Bureau of Reclamation, 1938, p. 44.

7 Treatise on Dams, Chapter 10 Arch Dams, Bureau of Reclamation, 1955.

elements and 10 to 12 cantilever elements. This time also includes a computation of the approximate concrete volume based on areas of horizontal sections through the dam. It is very easy, therefore, to compare the volume of trial layouts for a proposed structure.

A modification of this program was used to compute the following layout at joints and mid-blocks for Flamingo Gorge Dam at the top of each 7-1/2-foot construction lift:

1. Radial distance from axis to upstream and downstream faces of the dam
2. Coordinates of points at the faces of the dam referred to the base grid

TABLE 2.—PARTIAL LIST OF PROGRAMS IN USE

Function and File No.	Data required	Output
Geometry:		
182	General data at plane of centers, see Figure 1. Unit data 4 words: 2 aboutment angles, identification, and elevation	Arch and cantilever thickness, cantilever properties
Unit Cantilever Loads:		
101	Load peak elevations, abutment constants, cantilever properties output from Program No. 182	Forces and deflections due to unit radial, tangential, and twist loads on cantilevers
Arch Constants and D-Terms:		
800	Abutment constants, identification, elastic properties and controls. Tabular values for abutment angle[5] (15, 35, or 75 cards per set	Data for programs: No. 642 (Symm) No. 530 (Non-Symm)
Unit Arch Loads (Symmetrical):		
642	Output from Program No. 800	Forces and deflections in symmetrical arch due to symmetrical loads
Unit Arch Loads (Non-Symmetrical):		
530	Output from Program No. 800 for left and right sides of the dam	Forces and deflections due to non-symmetrical arch loads

Each set of computations required approximately 3 sec of machine computation time.

CANTILEVER DATA

Unit loads, triangular in shape, are used in applying loads to the cantilever elements. Abutment unit deformations of cantilever, as well as arch elements, were based on the work of Fredrik Vogt.[6] The equations involved could readily be solved by the computer, but little if any net saving would result, since the amount of programing is relatively high compared to the volume of work done.

Programing has been concentrated on the more repetitive parts of the problem where the computer shows cost advantages as high as 15 to 1.

Using the cantilever data obtained by Program No. 182 (thickness and location) plus deformation constants determined manually, Program No. 101 computes radial deflection of each cantilever determined by the ordinary theory of cantilever beams with contributions from bending, shear, and foundation yielding included. It also computes tangential cantilever deflections, by shear detrusion only, because cross-stream bending of a dam is normally considered to be negligible. Angular rotations of cantilever elements due to unit twist loads are also computed. All data prepared by computer for use in making trial-load studies come from the machine in punched cards in floated decimal form as described earlier. They can then be listed by tabulating equipment for visual inspection and use in the adjustments. The adjustments are carried out

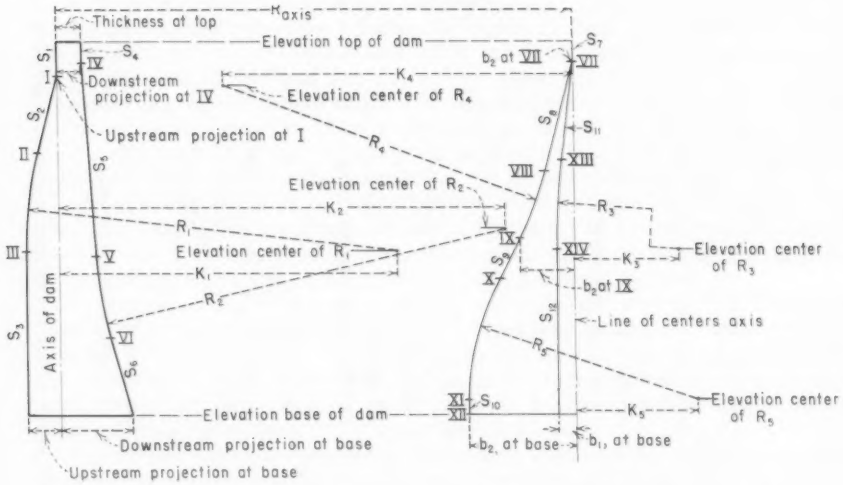

FIG. 1.—PLANE OF CENTERS LAYOUT FOR VARIABLE THICKNESS DAM

by use of mechanical calculators, using trial loads, plotting the results of each trial adjustment for graphical presentation and comparison.

ARCH DATA

The forces applied to arch elements during an analysis include radial, tangential, twist, and temperature loads. The machine program was set up to compute effects of temperature change with a linear variation from upstream to downstream faces of the arch in addition to the usual uniform temperature change. Other unit loads, symmetrical or nonsymmetrical, of forms indicated in Fig. 2, were programed, utilizing the tabular values f(X) that are given as Tables 10, and 17 through 21 of the USBR's Boulder Canyon reports.[6] (X is the angle from crown of arch to points where movements are to be computed.) The f(X) vlaues are listed for angular increments of 1° from 10° to 90° in the reference tables. Since the abutment angle seldom exceeds 60°, the tables

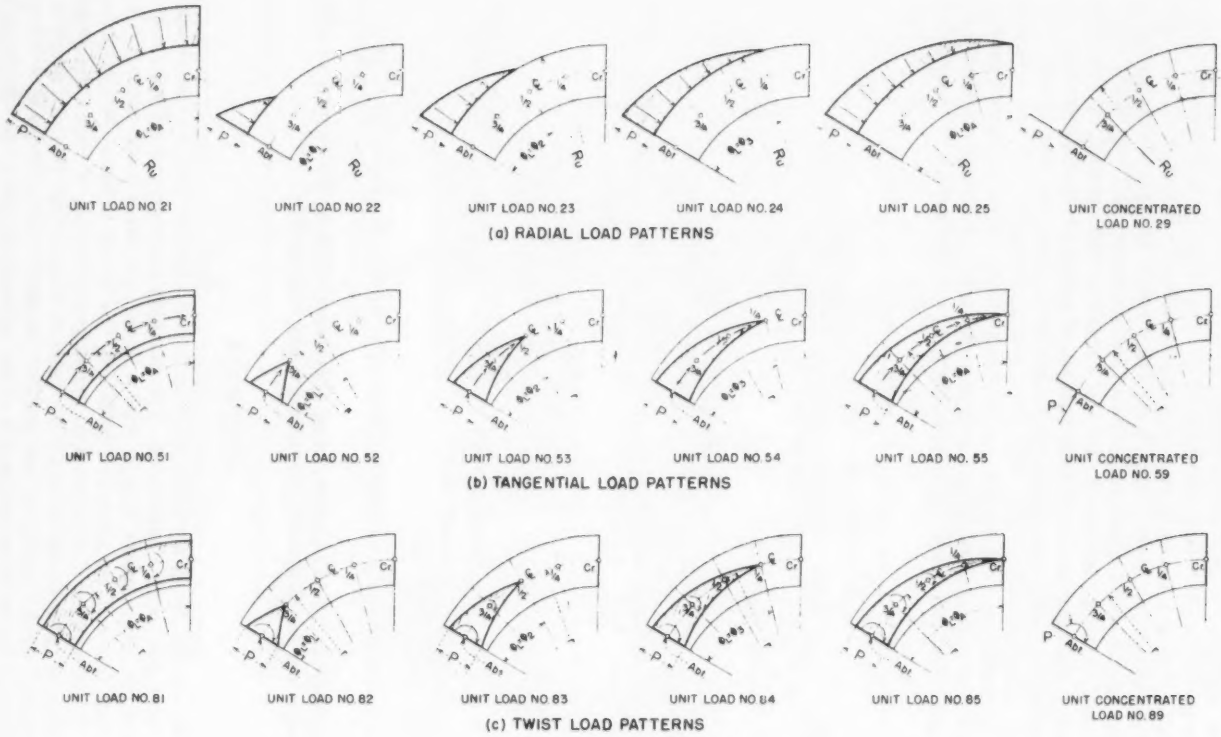
UNIT LOAD NO. 21
UNIT LOAD NO 22
UNIT LOAD NO. 23
UNIT LOAD NO. 24
UNIT LOAD NO. 25
UNIT CONCENTRATED LOAD NO. 29
(a) RADIAL LOAD PATTERNS
UNIT LOAD NO. 51
UNIT LOAD NO. 52
UNIT LOAD NO. 53
UNIT LOAD NO. 54
UNIT LOAD NO. 55
UNIT CONCENTRATED LOAD NO. 59
(b) TANGENTIAL LOAD PATTERNS
UNIT LOAD NO. 81
UNIT LOAD NO. 82
UNIT LOAD NO. 83
UNIT LOAD NO. 84
UNIT LOAD NO. 85
UNIT CONCENTRATED LOAD NO. 89
(c) TWIST LOAD PATTERNS

FIG. 2.—UNIT EXTERNAL LOAD PATTERNS SHOWING POSITIVE APPLICATION

were punched into sets of cards over the range of 10° to 60°. Each set consists of 75 cards that give f(X) data in the order that it is required by Program No. 800, which computes "arch constants" and "D-terms" due to unit forces and unit loads, respectively.

To permit the abutment angle to be chosen to the nearest 0.2° instead of nearest whole degree, that datum was used as a base for interpolation by Lagranian methods. The card library of f(X) now includes 251 sets of data from 10° to 60° at increments of 0.2°.

The arch constants and D-terms of Program No. 800 are based on rigid abutments and assume the arch cut at the crown. The second program of tandem set (642 if symmetrical and 530 if nonsymmetrical analysis) introduces effects of elastic abutment deformations and determines forces required at the arch crown to restore continuity in the arch element.

Input data required to compute the left or right half of an arch consist of 25 "words" plus cards of f(X) of the abutment angle. The following pertinent information is included in the heading 25 words of data: a) Thickness of arch one-quarter points; b) Upstream radius; c) Unit arch abutment movements; d) Coefficient of thermal expansion; e) Concrete modulus, E_c; f) Rock modulus, E_r; g) Abutment angle; h) Identification word; and, i) Controls to indicate what loads are to be worked, extent of analysis, and whether left or right side is being computed. The program checks to be sure the f(X) data set matches the abutment angle and will not run unless they are equal.

The last two digits of the identification word indicate the load code. For example, load 02 is a uniform temperature change of +1.0° F, and triangular radial loads from abutment to arch crown point would be numbered 15, 25, or 35 if applied to left side, both sides, or right side, respectively.

In computing data for a symmetrical analysis, the left side of the dam is usually studied and output from Program No. 800 is accepted without modification by Program No. 642 to compute arch deflections and forces due to symmetrical loads. A sample listing of data for a symmetrical radial load on an arch element of Dam B(1) is given in Table 3. Plan, profile, and section on plane of centers, of Dam B(1), are given on Fig. 3.

To compute data for a nonsymmetrical arch requires that the arch constants and D-terms for the two halves of the arch be determined individually by Program No. 800. The output data are merged in the following prescribed order to form input data for Program No. 530. (Side refers to the side of the dam looking upstream.)

Cards No.	Side	Data
1-5	Right	General arch description and
1-5	Left	control information
6-11	Right	Arch constants
6-11	Left	Arch constants
12-15	Right	D-terms linear temperature variation
12-15	Left	load 03
16-20	Right	D-terms uniform radial load and
16-37	Left	triangular loads on left
Etc.		

A sample of data for a nonsymmetrical load on a symmetrical arch of Dam B(1) is given in Table 4. This could just as well have been computed for a completely nonsymmetrical arch, but this tabulation permits a check of results obtained for symmetrical compared to nonsymmetrical load. A comparison of movements or forces from Table 3 (Load 25) with corresponding forces from Table 4 agrees to about the seventh significant place. The data from Table 4 must be added for both sides of the arch to check against the data as given in Table 3.

SUMMARY AND CONCLUSIONS

A summary of the arch data which is prepared for use in trial-load studies and the amount of machine time which is required for each type of analysis is indicated in Table 5.

TABLE 3.—ARCH DATA FOR TRIAL LOAD ANALYSIS—SYMMETRICAL LOADS

Data Indentified by Locations as Follows

Moments in Locations 0945-0949 Thrusts in Locations 9055-0959

Shears in Locations 965-0969 Radial Movements in Locations 0975-0979

Tangential Movements in Locations 0985-0989 Slopes in Locations 0995-0999

0935 500 315 02 Units—pounds and feet Data in Floated Decimal Form

Locs.	Crown Pt.	1 Qtr. Pt.	Half Pt.	3 Qtr. Pt.	Abut. Pt.
0945 5	-2699412755	-2181335855	-6567206054	1791764755	5032693755
0955 5	7460708953	7286667653	6772663453	5942677753	4835433853
0965 5		1602077753	3129410053	4510738153	5681615753
0975 5	2550176747	2258226647	1508644547	6326121946	5322546145
0985 5	-1120000040	2131294646	3127492546	2314835046	2074559745
0995 5	1600000038	7937750644	1236974845	1105616145	3490540644
0935	500 315 25				
0945 5	-4905921855	-2177147054	6658000055	5017164055	-1527713056
0955 5	9252689454	9097012154	8877346554	8964378854	9714486354
0965 5		1154157254	5890207053	1669043954	-5514682054
0975 5	-1910828247	-2386784647	-2879415047	-2112762047	-4086266046
0985 5	-2760000041	-5532343045	-2592622946	-4776961446	-4167833346
0995 5	1600000039	9946936044	-4619880043	-1895275645	-1181494445

Where two programs are listed together (as 800-642), they operate in tandem, that is, the output of one program is the input for the other. Programs No. 800 and 530 each occupy nearly all of the available space on the drum. The use of these data in making trial-load studies and the over-all times required to complete various types of studies are given in a paper by Merlin D. Copen.[8]

The economy of cost and time which has resulted from the use of programs described herein has been very satisfactory. The programs have been carefully checked out against data which were prepared by other means and are completely reliable. If a set of computations is performed twice on the computer, the output results will agree in every digit position.

[8] "The Design of Arch Dams by Use of the Trial-Load Method of Stress Analysis," by Merlin D. Copen.

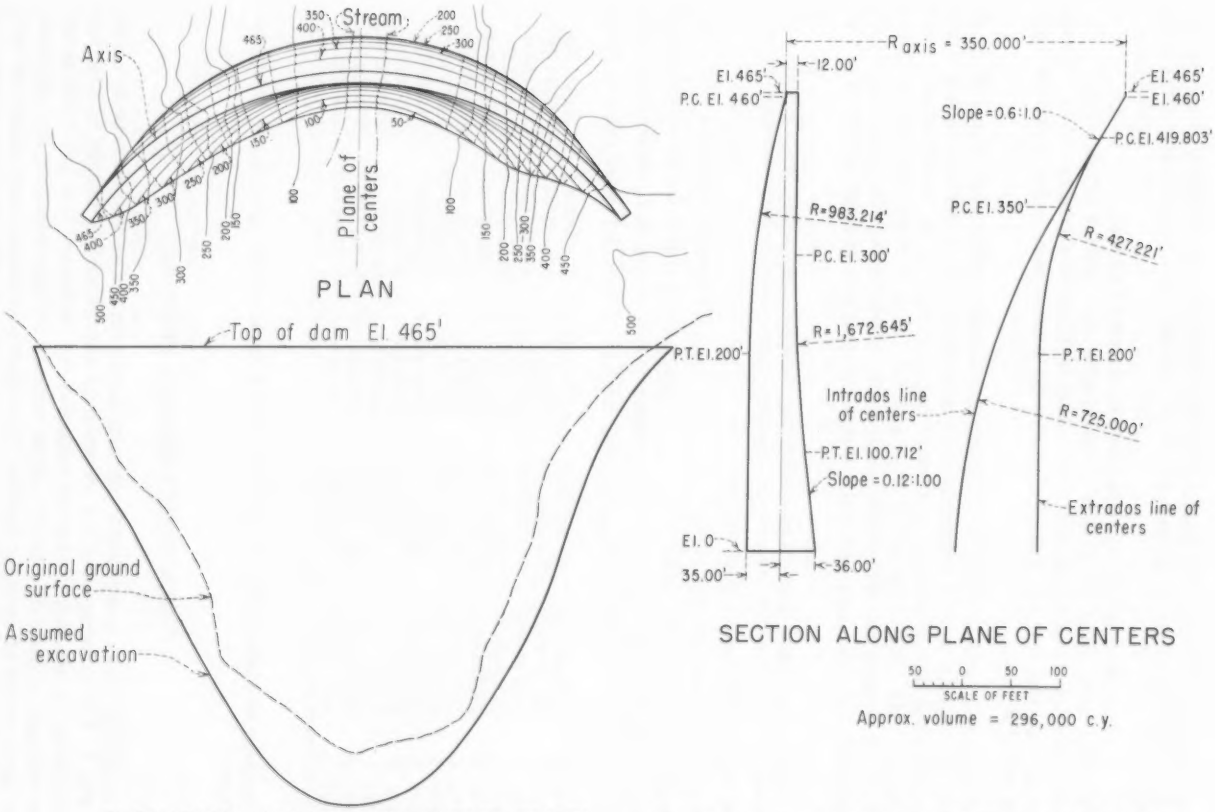

FIG. 3.—DAM B-1 PLAN, ELEVATION, AND SECTION

The programs that are in use, and that have been described herein, are fairly general but have restrictions, as follows:

1. Upstream (extrados) radius must be constant from abutment to abutment.
2. Abutment angle must be between 10° and 60° with permissible increment of 0.2°.
3. Contact of concrete to rock must be assumed radial.
4. Unit arch loads are linear between arch one-quarter points, Fig. 2.
5. Cantilevers are assumed to be radial to the arch extrados center. Thus, they are warped (except at crown) unless the line of centers for upstream face is a vertical line.

These restrictions apply to the programs as now constituted. Some of them could be relaxed without extensive reprograming. The upstream radius could change at the line of centers and would require only minor modification of the program. The tabular values of angles below 10° or greater than 60° could be developed without excessive additional work, if desirable. However, to obtain programs that are more general and that will produce results easier to use in

TABLE 4.—ARCH DATA FOR TRIAL LOAD ANALYSIS—NONSYMMETRICAL

Data Identified by Locations as Follows

Moments in Locations 0940-0949 Thrusts in Locations 0950-0959

Shears in Locations 960-0969 Radial Movements in Locations 0970-0979

Tangential Movements in Locations 0980-0989 Slopes in Locations 0990-0999

0935	500 315 15	Units—pounds and feet			Data in Floated Decimal Form
0940 R	-2452962055	-8705816555	-8237022055	-1109042055	1225629456
0945 L	-2452962055	8488099755	1489502056	6126206055	-2753342756
0950 R	4626344854	4846303354	4840155454	4608187954	4161223454
0955 L	4626344854	4250708954	4037191354	4356191154	5553263154
0960 R	-1526905854	-4978470053	5544393053	1580857954	2533520854
0965 L	1526905854	1652004254	3458140052	-3249901854	-8048203054
0970 R	-0554167046	1568630947	2306780447	1424722947	2045772346
0975 L	-9554121646	-3955415947	-5186196547	-3538485047	-6132038646
0980 R	2105938347	1816146547	1161183447	5324556946	1785301346
0985 L	2105935547	1760823447	9019213946	5475957045	-2382532046
0990 R	3928884645	2005008045	-6638168144	-2035984745	-8870514144
0995 L	3928884945	2999699945	-7100168044	-3931260645	-2068546145

a machine solution of the adjustment, a new series of programs are being developed.

NEW SERIES OF DATA PROGRAMS

An arch section, composed of circular segments and wedge sections, is shown in Fig. 4. The drawing does not necessarily represent the manner in which an arch would be shaped, but rather indicates the variables that are considered. For example, a parabolic arch could be studied by approximating its upstream face by a series of circular curves. The upstream radius may change at each point of intersection of cantilever with each element. The arch is composed of

TABLE 5.—SUMMARY OF ARCH DATA PREPARED

Program No.	Data	Machine time (approx. Hrs.)
182	Thickness and volume	0.3
	For Crown Adjustment Analysis	
101 800-642	Crown cantilever data 4 unit loads per arch	0.1 0.1/arch
	Symmetrical Radial Adjustment Analysis	
101 800-642	Cantilever data one side 8 unit loads per arch	0.2 0.15/arch
	Nonsymmetrical Radial Adjustment Analysis	
101 800-530	All cantilever data 13 unit loads per arch	0.3 0.5/arch
	Symmetrical Complete Analysis	
101 800-642	Cantilever data one side 20 unit loads per arch	0.2 0.3/arch
	Nonsymmetrical Complete Analysis	
101 800-530	All cantilever data 35 unit loads per arch	0.3 1.1/arch

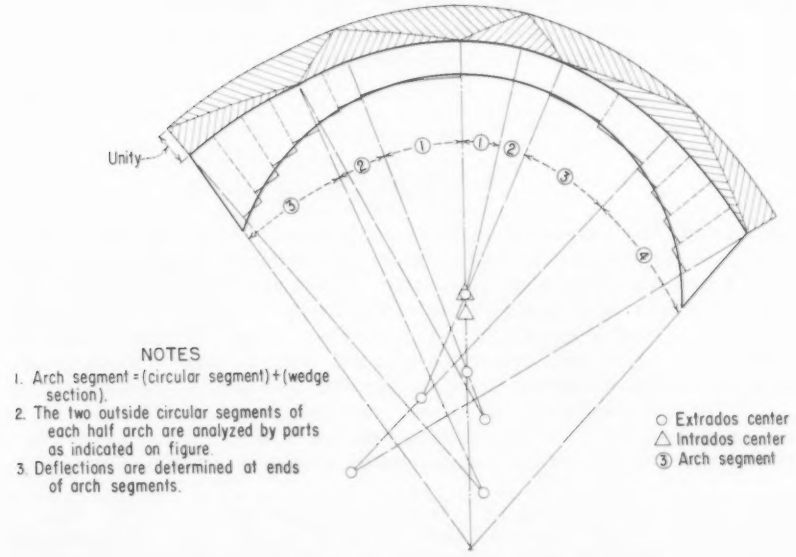
Unity
NOTES
1. Arch segment = (circular segment) + (wedge section).
2. The two outside circular segments of each half arch are analyzed by parts as indicated on figure.
3. Deflections are determined at ends of arch segments.
○ Extrados center
△ Intrados center
③ Arch segment

FIG. 4.—ARCH SECTION COMPOSED OF CIRCULAR SEGMENTS AND WEDGE SECTIONS

segments between cantilever points. The USBR plans to provide for a maximum of nine arch segments in each half arch.

Arch loads will peak at cantilever points as indicated in Fig. 4. Deflections will also be computed at these points. Because arch and cantilever deflections will be known at common intersection points, they may be compared directly by the computer, thereby making it easier to set up a procedure to obtain a machine process for trial adjustments.

Arch segments (Fig. 4) will be limited to 18.1° and f(X) will be computed from the power series of X in radians. The limit is set so that X squared is less than 0.1.

Much of the extensive checking that will be required can be done by the machine, choosing arch elements that can be computed using the new as well as the previously described programs.

Although not committed to any particular solution for the deflection adjustments, serious consideration will be given to an iteration process as opposed to a procedure based on the solution of a large group of simultaneous equations. Even after the new, more general programs are developed, it is expected that the programs described herein will be continued in use for crown adjustment studies and perhaps also for studies that are carried through the radial adjustment.

ACKNOWLEDGMENTS

The programs referred to in this paper were developed by the Bureau of Reclamation, Commissioner's Office, Denver, Colorado. Complete descriptions of them have not been prepared. They may be made available on an exchange basis at a later date after they have been completed.

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POWER DIVISION
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DESIGN OF ARCH DAMS BY TRIAL LOAD METHOD OF ANALYSIS

By Merlin D. Copen,[1] F. ASCE

SYNOPSIS

A brief resume of the basic principles involved in the trial-load method of stress analysis is presented. The various types of analyses, and their advantages and limitations, are examined. The application of trial-load methods to design problems and practical examples resulting from these procedures are illustrated and described in detail. It is also shown that the use of the trial-load method of stress analysis makes possible economy not only in the design of arch dams, but also in the time and expense required to produce such designs.

INTRODUCTION

Since the end of World War II, interest in the design and construction of dams has received considerable impetus. This interest has resulted in novel approaches to problems of design. Currently the methods used appear to fall in one or more of the following categories: (1) analysis of small scale models; (2) thin cylinder theory; (3) relaxation methods; (4) shell theory; and (5) trial-load analysis. Each of these has advantages and limitations. The choice of methods generally resolves into accuracy and reliability desired as opposed to the time, finances, and experience available for design procedures.

After exhaustive study of the various possibilities, the United States Bureau of Reclamation, Dept. of Interior (USBR) adopted the trial-load method of stress

Note.—Discussion open until January 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 4, August, 1960.

[1] Engr., Bur. of Reclam., U. S. Dept. of Interior, Denver, Colo.

analysis for designing and analyzing arch dams. Whereas there have been notable advances in the use of other methods, the USBR has still found the use of trial-load to be completely satisfactory and unexcelled in this field. Recent developments in the use of electronic digital computers,[2] and the effective application of simplified analyses have made this method even more effective.

No attempt will be made in this paper to justify the basic concepts involved in trial-load theory because this has been done in previous publications.[3,4] It is the intent of the author to present a basic background of concepts involved in trial-load analysis, describe their application to design, and illustrate the practical aspects of the use of trial-load methods in the design of arch dams.

BASIC PRINCIPLES

The trial-load method of stress analysis assumes that the load applied to an arch dam would be divided between horizontal and vertical elements in such a way as to produce equal movements in all directions, at points of intersection of these horizontal and vertical elements. Because the required agreement of all deformations can best be obtained by assuming different distributions of load and computing the resulting movements until the specified conditions are fulfilled, the procedure is logically called the trial-load method.

It may be assumed that the dam to be analyzed is divided into a series of arch and cantilever elements by passing through it a series of horizontal and vertical planes. The horizontal planes defining the arch elements are assumed to be spaced a unit distance apart, and the vertical radial planes defining the cantilever elements spaced a unit distance apart at the axis of the dam. The sum of these arch elements occupies the total volume of the dam, which is also the case with the cantilever elements. Each arch and each cantilever is assumed to move independently of all others, but at the conclusion of the analysis, geometrical continuity must be restored at all points in the structure.

Instead of investigating a large number of horizontal and vertical elements, only a relatively few representative arches and cantilevers are studied to complete the analysis within a reasonable length of time. If the dam is approximately symmetrical about the maximum cantilever section, only half of the structure need be analyzed, and five to seven arches and four to seven cantilevers may be sufficient. If the structure is nonsymmetrical, however, both sides must be analyzed and approximately twice as many cantilevers are necessary.

The trial-load analysis is carried out in steps, generally referred to as adjustments. Three adjustments, radial, tangential, and twist, are necessary to achieve geometrical congruence. The radial adjustment accounts for radial displacement. Tangential movement, vertical displacement, and rotation about the radius are brought into agreement by use of the tangential adjustment. The twist adjustment provides rotational congruency about the tangential and vertical axes. When equality of these three linear and three angular displacements

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2 "Use of Electronic Digital Computer to Prepare Data for Trial Load Analysis of Arch Dams," by L. R. Scrivner, presented at the Spring Convention of ASCE in New Orleans, La., March 7-11, 1960.

3 Boulder Canyon Project Final Reports, Part V, Bulletin 1, "Trial Load Method of Analyzing Arch Dams," Bur. of Reclam., U. S. Dept. of the Interior, Denver, Colo.

4 "Treatise on Dams," Chap. 10, Arch Dams, Design Standards No. 2, Bur. of Reclam., U. S. Dept. of the Interior, Denver, Colo.

of the arch with those of the cantilever has been achieved at their points of juncture, the requirements for a solution are complete.

The loads on the arch and cantilever elements that produce deformation agreement may be freely chosen with the provision that the sum of the arch and cantilever loads must equal the external load at every point. The external loads include all external forces acting on the dam. These consist of the weight of the structure, reservoir water, tail water, temperature changes, earthquake shocks, and silt and ice loads. These loads are divided between the arch and cantilever system until a satisfactory agreement of radial deflections is obtained.

To complete the deformation agreement, it is necessary to introduce internal tangential and twist loads. These are applied in pairs of equal and opposite loads, one acting against the arch and the other against the cantilever. By this means, arch and cantilever deflections may be brought into tangential and rotational agreement without changing the external load on the structure. The internal loads represent forces set up by the interaction between the assumed arches and cantilevers in the dam.

To facilitate the process of dividing the loads between the horizontal and vertical elements, certain patterns of loads called unit loads have been developed. In the case of the arches, these loads consist of a uniform load over the entire length of the arch and triangular loads varying from a maximum value at the abutment to zero at the quarter points. These loads may be used to represent radial forces, tangential thrusts, or moments, depending on the adjustment under consideration. The unit cantilever loads are assumed to vary from a maximum at one arch elevation to zero at the arches directly above and below. These loads are used to represent radial shears, tangential shears, or twisting moments on the cantilevers, depending on the appropriate adjustment.

In applying the unit loads, it is advantageous to compute the movements of arch and cantilever elements produced by the unit loads before attempting to divide the external load between the arches and cantilevers. The arches are statically indeterminate structures terminating at elastic abutments. Computations of deflections of arch elements are made by the elementary theory of flexure for curved beams, with the effects of rib-shortening, transverse shear, and yielding abutments included. The arch elements resist radial forces applied at the faces, tangential forces and horizontal moments applied at the center lines, and twisting moments in vertical radial planes. The cantilevers are elastic units, seated on an elastic foundation. They resist vertical and radial forces applied to the upstream or downstream faces, and tangential forces, twisting and bending moments applied at the centerlines.

The total loads resisted by horizontal and vertical elements are determined by the trial-load adjustments. With these loads, stresses may be computed throughout the dam, provided a definite variation of stress between the upstream and downstream faces of the dam is assumed. Three of the stresses, the vertical stress normal to a horizontal plane, the horizontal stress normal to a vertical radial plane, and the horizontal tangential shear stress acting on a horizontal plane, are assumed to vary linearly between the upstream and downstream faces. A nonlinear distribution of stress across the dam might be assumed, in which case it would be necessary to make a new series of adjustments based on such an assumption. By means of the linear assumption, the stresses previously mentioned may be computed using total arch and cantilever loads. From these stresses, principal stresses or stresses on any plane may be computed throughout the dam.

TYPES OF TRIAL-LOAD ANALYSES

Trial-load analyses may be classified according to their relative accuracy and corresponding complexity. Progressing from the simplest to the most comprehensive, these analyses are called crown cantilever analysis, radial deflection analysis, and complete trial-load analysis.

A crown cantilever analysis consists of an adjustment of radial deflections at the crown (usually the maximum) cantilever with the corresponding deflections of the crowns of the arches. This type of analysis assumes a uniform distribution of radial load from the crowns of the arches to their abutments, and neglects the effect of tangential shear and twist. Although the results obtained from this analysis are rather crude, the analysis has the advantage of requiring a very short time to complete, and when used with judgment, is a very effective tool for the preliminary design of arch dams.

A radial deflection analysis is one in which radial deflection agreement is obtained at arch quarter points with several representative cantilevers by an adjustment of radial loads between these structural elements. With the use of this type of analysis, loads may be varied between the crowns and abutments of arches, thus producing a more realistic distribution of load in the dam. The effects of nonsymmetry may also be included in this analysis. The time required to complete a radial deflection analysis is only slightly greater than that necessary for a crown cantilever analysis. Because the effect of tangential shear and twist are neglected in this analysis, the results are not complete but do furnish a much better estimate of the stresses than is possible from a crown adjustment.

With the complete trial-load analysis, agreement of three linear and three angular displacements is obtained by properly dividing the radial, tangential, and twist loads between the arch and cantilever elements. The accuracy of this analysis is limited only by the exactness of the basic assumptions, the number of horizontal and vertical elements chosen, and the magnitude of error permitted in the slope and deflection adjustments. The results from complete analyses have been confirmed by model tests.[5] The major limitation to the use of the complete trial-load analysis is the time required to perform such studies, and the high degree of technical training necessary to efficiently conduct such an analysis. The time element has been effectively reduced by use of the electronic computer, and will be further diminished as the analysis is completely programmed. The number of highly-trained engineers required will also be greatly reduced.

APPLICATION OF TRIAL-LOAD METHODS TO DESIGN PROBLEMS

The design of arch dams, as presented in this paper, is based on the stress conditions determined in the structure and governed by the established design criteria. This assumes, of course, that site conditions are satisfactory and other requirements, such as diversion, spillway, penstocks, powerhouse, and others, have or may be satisfied. The object of the design studies is, therefore, to obtain a maximum efficiency from the concrete in the dam within the limits imposed by design criteria.

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[5] Boulder Canyon Project Final Reports, Part V, Bulletin 3, "Model Tests of Boulder Dam," Bur. of Reclam., U. S. Dept. of the Interior, Denver, Colo.

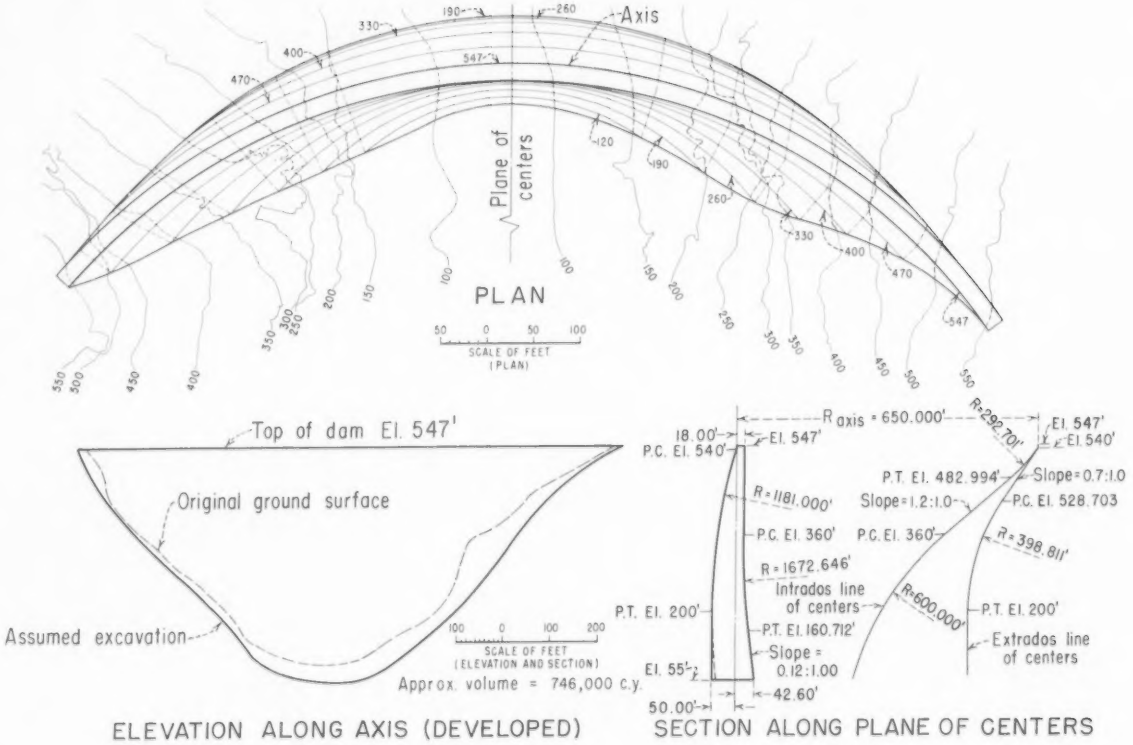

FIG. 1.—DAM A PLAN, ELEVATION, AND SECTION

At present, the USBR limits the maximum compressive stress in the dam to 1,000 psi for the most severe loading conditions expected to occur at the site. These loads would usually consist of normal reservoir water, the maximum earthquake shock that might occur in the area, the mean minimum temperature conditions anticipated, and any tail water, silt, ice or backfill material which will be present at the site. A limited amount of tensile stress is permitted in the dam, the magnitude and location to be tolerated being determined for each individual situation.

The initial layout for an arch dam is based largely on the experience and judgment of the designer. He may use such tools as thin cylinder theory, independent arches, or previous similar designs to determine the shape and properties of the section to be used. The abutments are designed to require a minimum excavation to sound rock, and still provide ample stability and a smooth profile along the plane of contact with the dam. As soon as a layout for the particular site has been completed, it is checked by means of a crown cantilever analysis to obtain an estimate of the stresses in the proposed dam. Currently the deflections of the arches and cantilevers produced by unit loads are computed by an electronic digital computer. The time required, in a normal situation, to determine stresses with a crown adjustment, is approximately three days for two men. Several layouts may be necessary before a satisfactory stress condition is obtained. Then a radial deflection adjustment is made. This provides a more complete stress picture and might indicate the possibility of necessary or desirable changes. The radial deflection analysis requires approximately two days more than the crown cantilever analysis, or a total of approximately five days for two men.

In practice, after a design has been analyzed and found to be acceptable with a radial deflection analysis, the effects of tangential shear and twist are estimated, based on the experience of the designer. If the layout appears to be satisfactory, other engineers proceed with necessary details and appurtenances. Now the final test of the efficiency of the dam is made. While the detailed design work proceeds, a complete trial-load analysis is made of the structure. This will require from approximately 100 to 150 man-days, depending on the size and complexity of the dam and the accuracy required from the analysis. It is anticipated that in the near future, further application of electronic computer processes will result in considerable reduction in this time. If this analysis indicates that improvements can be made in the layout, such changes are made and incorporated in the detailed design procedures.

To demonstrate the application of trial-load methods to the design of arch dams, three sites have been selected representing a wide V-shaped canyon, a U-shaped canyon, and a narrow V-shaped canyon. Drawings and tables showing layouts and results from various types of trial-load studies have been prepared to provide examples of typical design studies.

The examples are chosen also to show three phases of development in the design. First, Dam A shows an advanced design with the various types of analyses leading to a complete analysis. Second, Dam B is a feasibility-type study with only a crown cantilever analysis but a well developed design. Third, Dam C is a preliminary-type study with a simple analysis, one which will need further refinement.

Fig. 1 shows a plan, elevation, and section along the plane of centers of a dam in a wide V-shaped canyon, designated Dam A. The crest length of the dam is approximately 1,200 ft and the maximum height is 490 ft. The loading

conditions for which this dam was designed include the following: normal reservoir water surface, elevation 540; an earthquake shock having an acceleration of one-tenth gravity with a period of vibration of 1 sec; and the effects of mean minimum operating temperatures. In Table 1 the results of a crown cantilever analysis are given. Cantilever stresses are acting in a vertical direction, normal to horizontal planes. Arch stresses are acting in a horizontal direction, normal to vertical radial planes.

Probably the most efficient design of an arch dam is one in which the critical compressive areas, that is, the crown extrados and abutment intrados, are subjected to the maximum allowable stress without producing excessive tensile stresses in other parts of the dam. An inspection of Table 1 indicates that in both of these areas most of the compressive stresses approach or exceed 1,000 psi. The maximum arch tensile stress of 197 psi would be considered excessive if all factors affecting the stresses had been included in this analysis. Both

TABLE 1.—DAM A: CROWN CANTILEVER ANALYSIS

Elevation	Arch stresses[a]				Crown cantilever stresses[a]	
	Abutment		Crown		Upstream face	Downstream face
	Extrados	Intrados	Extrados	Intrados		
547	+366	+ 664	+ 594	+433	0	0
470	+255	+ 915	+ 888	+467	+173	- 38
400	+ 92	+1,027	+1,080	+373	+243	+ 8
330	- 41	+1,031	+1,171	+224	+275	+ 94
260	-147	+ 988	+1,133	+ 38	+277	+ 205
190	-197	+ 837	+ 943	-113	+217	+ 373
120	-148	+ 588	+ 618	-151	+ 21	+ 671
55					-338	+1,136

\+ indicates compression.
\- indicates tension.

[a] All stresses in pounds per square inch.

the maximum compressive and tensile stresses computed in the crown cantilever exceed the allowable limits. Based on the designer's estimate of those factors that are not evaluated in this study, however, the design is considered adequate and a radial deflection analysis is made.

Table 2 shows the arch and cantilever stresses, parallel to the faces of the dam, resulting from a radial deflection analysis. Considerable change from the crown cantilever analysis is indicated in the abutment area of the arch elements. This is due primarily to the change from a uniform to a variable load distribution between the crowns and abutments of the horizontal elements. Stresses were also determined, by use of a radial deflection analysis, for normal reservoir water surface but without the effects of earthquake. The results of this study are shown in Table 3. Substantial reductions in the maximum stresses are indicated when the earthquake effects are omitted. The high tensile stress at the upstream face of the base of the crown cantilever changes from 345 to 225 psi, a reduction of more than 30%.

The results of a complete trial-load analysis of Dam A are shown in Figs. 2 and 3. Arch and cantilever stresses parallel to the faces of the dam are shown

TABLE 2.—DAM A: RADIAL DEFLECTION ANALYSIS

Elevation	Arch Stresses[a]									
	Abutment		3/4		1/2		1/4		Crown	
	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados
547	+600	+ 298	+316	+585	+435	+465	+550	+349	+ 560	+ 338
470	+424	+ 570	+306	+757	+422	+693	+756	+389	+ 981	+ 172
400	+303	+ 669	+167	+921	+449	+731	+926	+314	+1,154	+ 108
330	+ 63	+ 870	+146	+909	+511	+654	+972	+277	+1,180	+ 104
260	-112	+1,037	+160	+839	+571	+515	+966	+198	+1,134	+ 62
190	-171	+ 915	+105	+653	+466	+327	+800	+ 34	+ 939	- 88
120	-133	+ 614	+ 95	+368	+342	+122	+540	- 66	+ 618	- 137

Elevation	Cantilever stresses[a]									
	A		B		C		D		Crown	
	Upstream face	Down-stream face	Upstream face	Down-stream face	Upstream face	Down-stream face	Upstream face	Down-stream face	Upstream face	Down-stream face
547	0	0	0	0	0	0	0	0	0	0
470	+134	- 22	+134	- 8	+143	- 8	+151	- 12	+ 166	- 24
400	+127	+ 72	+123	+ 99	+146	+ 94	+188	+ 63	+ 236	+ 20
330			+ 51	+271	+107	+237	+202	+162	+ 268	+ 104
260			- 20	+448	+ 65	+382	+204	+268	+ 267	+ 217
190					+ 24	+523	+166	+410	+ 207	+ 388
120					- 54	+700	+ 20	+658	+ 12	+ 690
65							-204	+971	—	—
55									- 345	+1,159

[a] All stresses in pounds per square inch. See Figure 2 for location of arch and cantilever points. Effect of earthquake included.
+ indicates compression
- indicates tension

TABLE 3.—DAM A: RADIAL DEFLECTION ANALYSIS

Elevation	Arch stresses[a]									
	Abutment		3/4		1/2		1/4		Crown	
	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados
547	+424	+185	+198	+413	+303	+306	+390	+218	+361	+ 248
470	+323	+444	+233	+586	+334	+525	+582	+300	+752	+ 136
400	+242	+559	+143	+753	+379	+592	+758	+264	+941	+ 99
330	+ 55	+734	+124	+767	+431	+553	+821	+235	+997	+ 88
260	- 87	+859	+124	+713	+469	+440	+812	+162	+959	+ 42
190	-142	+783	+ 93	+560	+401	+283	+686	+ 33	+805	- 70
120	-112	+534	+ 85	+321	+299	+109	+471	- 54	+538	- 115

Elevation	Cantilever stresses[a]									
	A		B		C		D		Crown	
	Upstream face	Down-stream face	Upstream face	Down-stream face	Upstream face	Down-stream face	Upstream face	Down-stream face	Upstream face	Down-stream face
547	0	0	0	0	0	0	0	0	0	0
470	+130	- 19	+132	- 8	+141	- 8	+144	- 8	+153	- 14
400	+141	+ 54	+143	+ 74	+161	+ 73	+188	+ 57	+222	+ 27
330			+102	+209	+149	+187	+212	+143	+264	+ 99
260			+ 52	+359	+129	+305	+229	+231	+283	+ 190
190					+101	+431	+210	+354	+248	+ 334
120					+ 37	+590	+ 91	+573	+ 87	+ 600
65							-104	+854	—	—
55									-225	+1,021

[a] All stresses in pounds per square inch. See Figure 2 for location of arch and cantilever points. Earthquake effects are not included.
+ indicates compression
- indicates tension

LOADING CONDITIONS

Reservoir water surface elevation 540'
Temperature of concrete at time of grouting contraction joints assumed to be 38° Fahrenheit
Temperatures used in analysis are changes between average arch temperatures at time of joint closure and mean minimum operating temperatures.
Earthquake assumptions:
Dam moves up and downstream horizontally in the direction of the line of centers. increased water pressure acts equally on all cantilevers. Period of vibration = 1.0 second
Acceleration = 0.1 gravity. Effects of vertical accleration neglected.
Weight of concrete = 150 pounds per cubic foot
Modulus of elasticity of concrete = 2,500,000 pounds per square inch
Modulus of elasticity of foundation rock = 2,000,000 pounds per square inch
Poisson's ratio for concrete and abutment rock = 0.20
Coefficient of thermal expansion for concrete = 0.000,005,6 per degree Fahrenheit.

EXPLANATION

All arches analyzed as symmetrical.
Arch stresses are acting in horizontal directions parallel to the edges of the arches
E = Stress at extrados of arch
I = Stress at intrados of arch
S_R = Maximum arch shear stress
(+) indicates downstream shear
Cantilever stresses are acting parallel to the edges of the cantilevers.
U = Stress at upstream edge of cantilever
D = Stress at downstream edge of cantilever
S_C = Maximum cantilever shear stress
(+) indicates downstream shear.
All stresses are in pounds per square inch.
+ = Compression. − = Tension
□ = Crown of arch.
○ = Intermediate arch points.

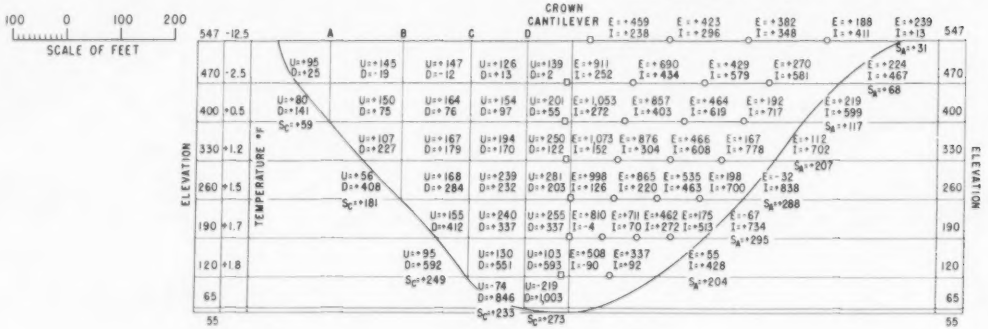

FIG. 2.—DAM A ARCH AND CANTILEVER STRESSES

α = Angle first principal stress (σP_1) makes with the vertical. Positive angle measured in a clockwise direction on the left side of the dam, and in a counter-clockwise direction on the right side of the dam.
σP_1 = First principal stress.
σP_2 = Second principal stress.
(+) Indicates downstream shear.

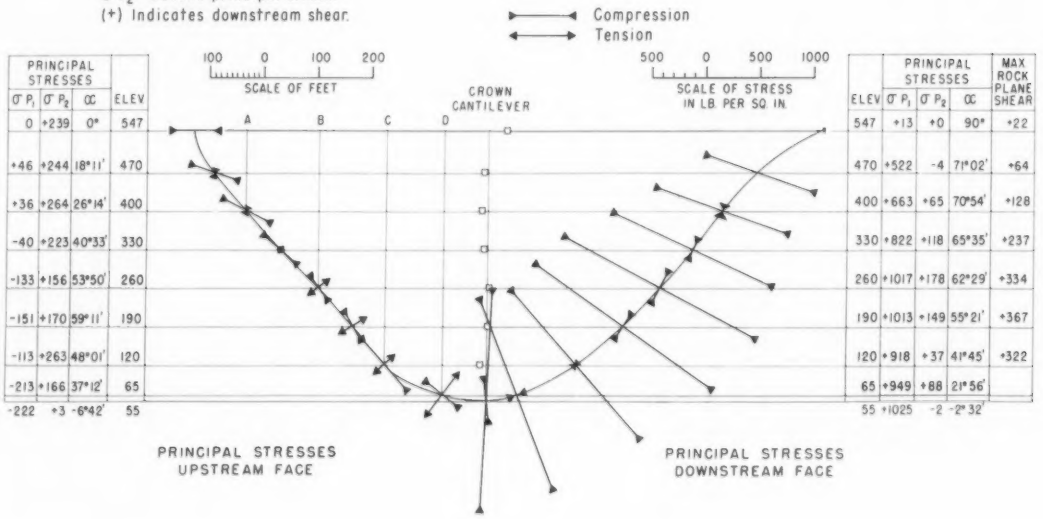

FIG. 3.—DAM A PRINCIPAL STRESSES

in Fig. 2, and principal stresses at the upstream and downstream faces of the abutment are plotted and tabulated in Fig. 3. An examination of these stresses indicates that the maximum allowable compressive stresses may be expected to occur in all of the abutment area except very near the top of the dam. Also, at the crowns of the arch elements, maximum compressions were found in all but the very highest and lowest points studied. The stresses in the top of the dam may be increased to near the maximum allowed by increasing the relative stiffness of this area of the dam. Because this is always the longest part of the dam, any increase in thickness results in a substantial addition to the concrete volume. Added stiffness in this area also results in larger tensile stresses on the downstream face of the cantilever elements. Therefore, it has been the practice of the USBR to keep the dam to a minimum thickness near the top. As is shown in the cantilever and principal stresses, the maximum allowable stress also occurs at the base of the dam. The tensile stresses computed along the abutment at the upstream face of the dam are higher than is desirable. These stresses may be substantially reduced or eliminated by making a change

TABLE 4.—DAM B (1): CROWN CANTILEVER ANALYSIS

Elevation	Arch stresses[a]				Crown Cantilever stresses[a]	
	Abutment		Crown		Upstream face	Downstream face
	Extrados	Intrados	Extrados	Intrados		
465	+121	+ 290	+251	+159	0	0
390	+151	+ 699	+591	+263	+126	+ 4
315	+ 37	+ 932	+846	+216	+191	+ 65
240	- 87	+1,019	+981	+ 88	+300	+ 81
165	-168	+ 937	+934	- 51	+411	+ 83
90	-192	+ 701	+699	-183	+333	+257
0					+ 18	+923

[a] All stresses in pounds per square inch.
+ indicates compression
- indicates tension

in the upstream face of the crown section, indicated by a dashed line on Fig. 1, accompanied by a slight reduction in extrados radii.

In Fig. 4 is shown a plan, elevation and section along the plane of the centers of a dam in a narrow V-shaped canyon, designated Dam B. This dam is 465 ft high with a crest length of approximately 650 ft. The design was based on the following loading conditions: normal reservoir water surface, elevation 460; an earthquake shock having an acceleration of one-tenth gravity with a period of vibration of 1 sec; the included effects of mean minimum operating temperature. In Table 4 are presented the results of a crown cantilever analysis made for this layout. Cantilever stresses are acting in a vertical direction normal to horizontal planes. Arch stresses are acting in a horizontal direction normal to vertical radial planes.

From these stresses it appears that economy can be achieved and the stress condition improved by thinning the crown section and reducing the radii to the

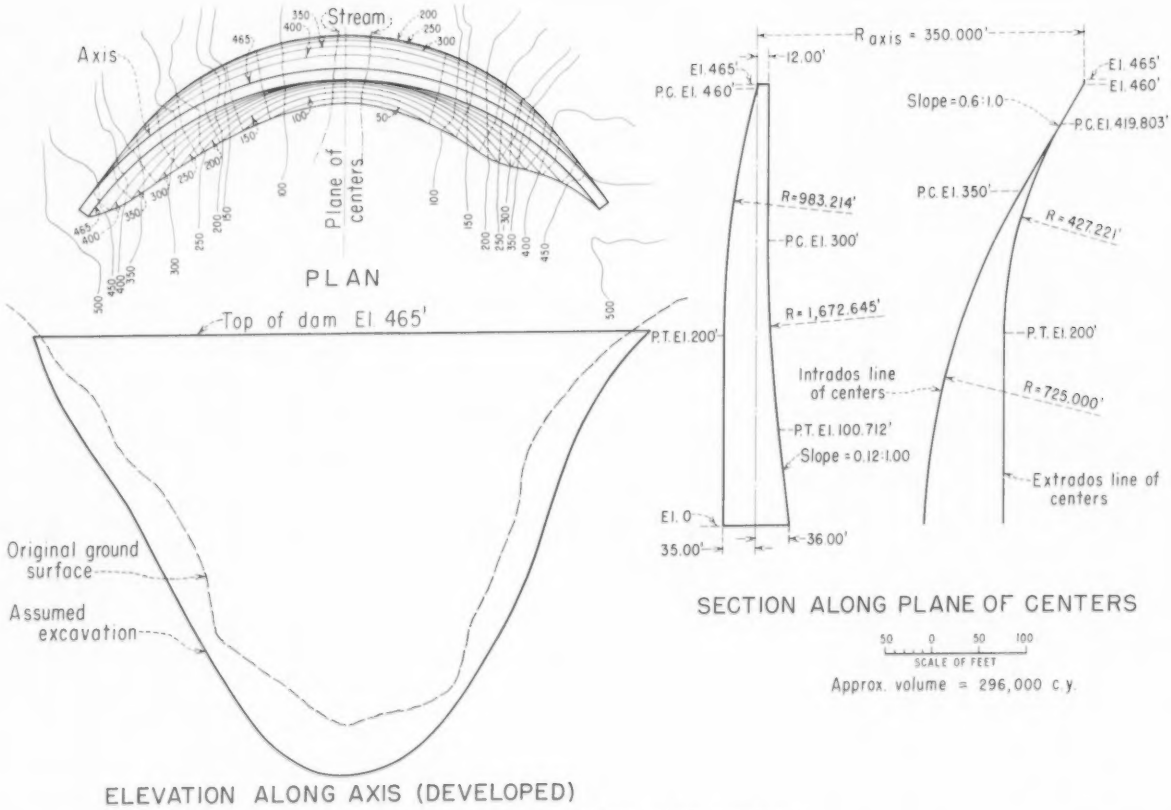

FIG. 4.—DAM B-1 PLAN, ELEVATION, AND SECTION

extrados. This was done and the resulting design is shown in Fig. 5. A stress analysis has not been made of this layout because, in the opinion of the designer, it is entirely adequate for preliminary design. The time required to complete this portion of the design of Dam B was less than ten man-days. An estimate of the effects of a complete trial-load analysis indicates that little, if any, change will be necessary in this design unless unexpected conditions are found as the site is developed. Radial and complete trial-load studies will be made to confirm this opinion as more complete data are available.

A plan, elevation and section along the plane of centers for a dam in a U-shaped canyon, is shown in Fig. 6. Dam C is 520 ft high and has a crest length of approximately 1,350 ft. The loading requirements for this site include reservoir water surface at elevation 540, an earthquake acceleration of one-tenth gravity with a period of vibration of 1 sec, and effects of mean minimum operating temperature. The results of a crown cantilever analysis of this design

TABLE 5.—DAM C: CROWN CANTILEVER ANALYSIS

Elevation	Arch stresses[a]				Crown cantilever[a] stresses	
	Abutment		Crown			
	Extrados	Intrados	Extrados	Intrados	Upstream face	Downstream face
560	+338	+ 695	+610	+420	0	0
500	+312	+ 909	+769	+444	+134	- 36
450	+228	+ 976	+835	+407	+207	- 35
400	+129	+ 996	+914	+369	+243	+ 4
350	+ 9	+1,027	+975	+284	+254	+ 69
300	-100	+1,009	+975	+165	+245	+ 155
250	-153	+ 906	+899	+ 66	+211	+ 264
200	-160	+ 763	+764	+ 1	+140	+ 406
150	-128	+ 592	+591	- 22	+ 28	+ 586
100	- 71	+ 426	+411	0	-130	+ 813
40					-385	+1,160

[a] All stresses in pounds per square inch.
+ indicates compression
- indicates tension

are shown in Table 5. Cantilever stresses are acting in a vertical direction normal to horizontal planes. Arch stresses are acting in a horizontal direction normal to vertical radial planes.

The possibility of increased economy and efficiency in the use of the concrete in this dam is indicated from the stress results. By reshaping and thinning the crown section in addition to reducing extrados radii, the volume of concrete will be reduced and the stresses will more nearly approach a maximum stress condition throughout the dam. Care must be taken, however, to keep both tensile and compressive cantilever stresses within allowable limits near the base of the dam. This layout is considered satisfactory for preliminary work but would require refinement and more complete analysis before the finished product is obtained.

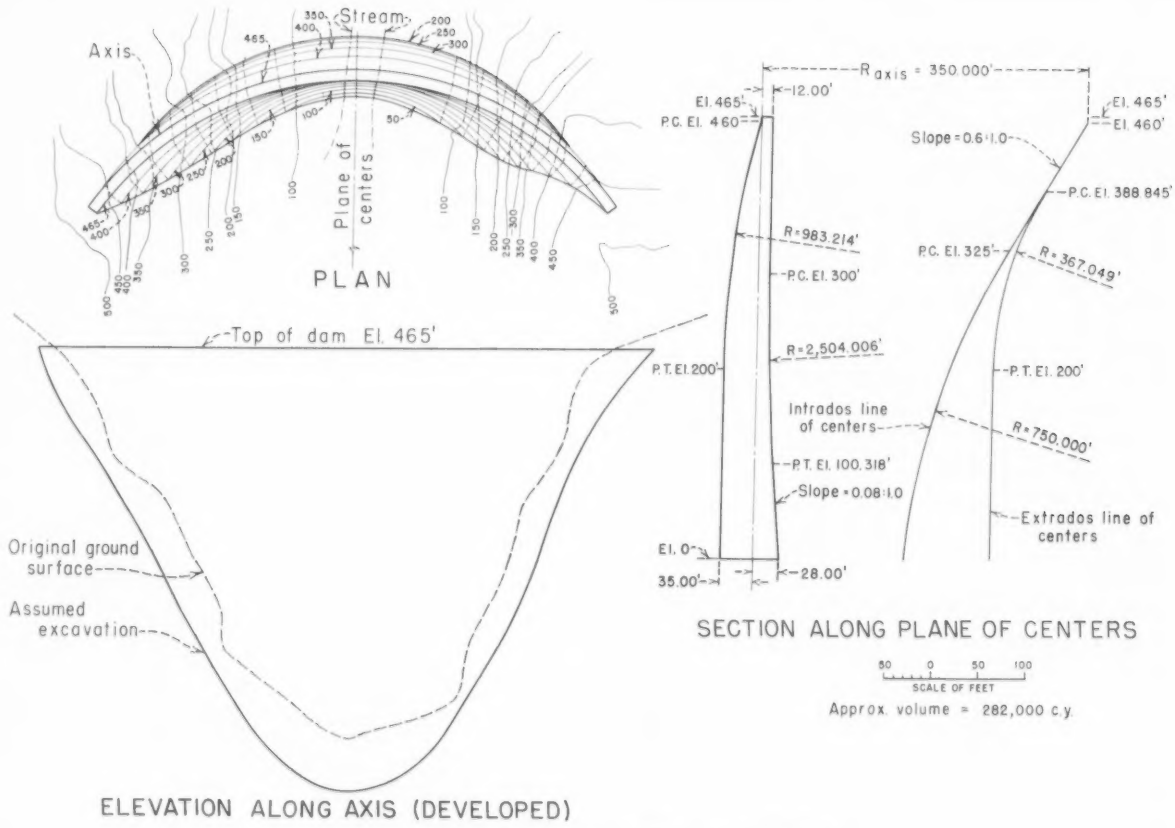

FIG. 5.—DAM B-2 PLAN, ELEVATION, AND SECTION

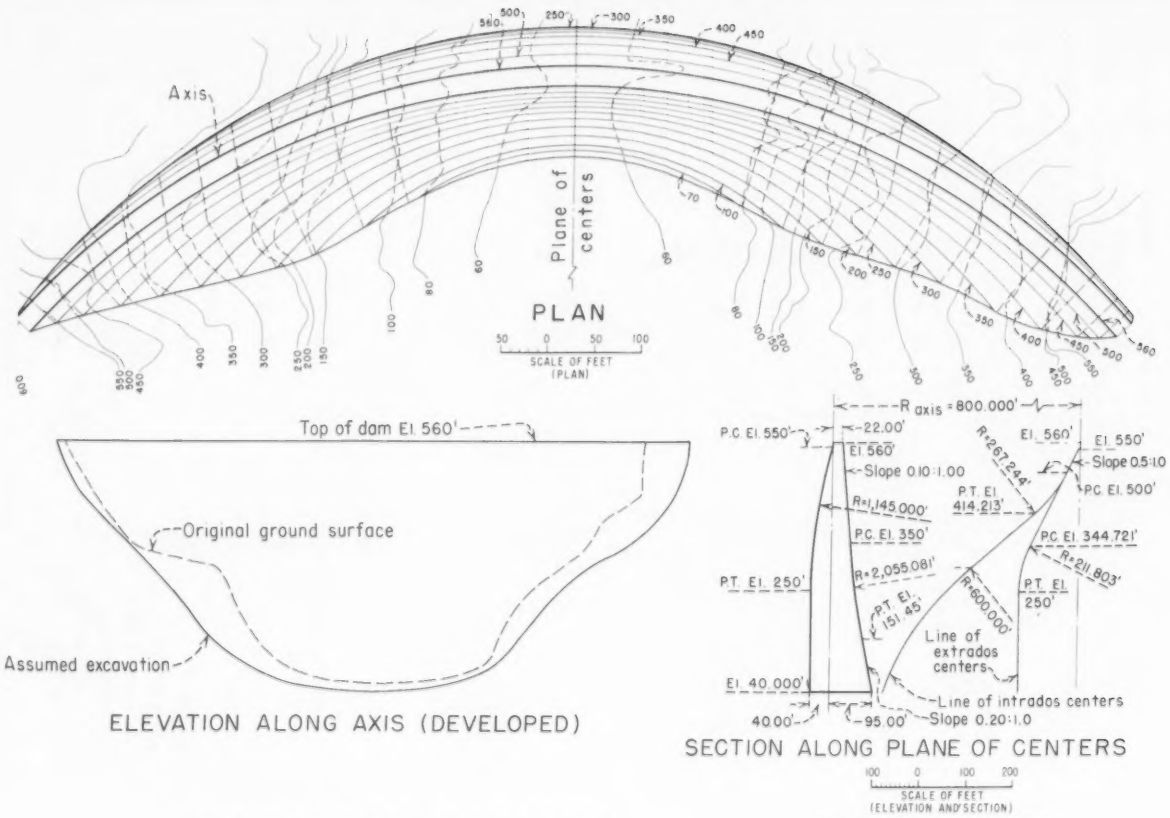

FIG. 6.—PLAN, ELEVATION, AND SECTION

CONCLUSIONS

In the design of arch dams, just as any other engineering structure, if a sound analytical solution is possible, it is generally preferable. Shortcuts and rough approximations as substitutes for rigorous solutions are not satisfactory and may result in unsafe designs. Small scale models, until recent years, have not been generally accepted as a means for design, but rather as a check on analytical processes. The possibility of error in the minute measurements necessary, in addition to the magnification of such errors when restored to the prototype, have always caused doubt regarding the reliability of model results. In the case of arch dams, some conditions, such as dynamic and thermal loads, are subject to analytical solution when model analyses of these conditions is not practical.

The trial-load method of analysis provides a completely satisfactory analytical means for designing arch dams. To use this method effectively, an understanding of the basic theory involved and its practical application is absolutely necessary. This type of analysis is applicable to any type of arch dam under any condition of load that may be expected to occur. The accuracy of the solution is limited only by the reliability of the assumptions required, the number of representative structural elements chosen, and the error permitted in the deflection and slope adjustments.

With the use of an electronic digital computer and simplified trial load methods, an experienced designer can prepare an efficient and economical design in a nominal period of time and check the results with complete confidence in their reliability.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN OF KARADJ HYDROELECTRIC PROJECT

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SYNOPSIS

This paper describes the development and design of a large water supply and hydroelectric project near Tehran, Iran. Also described are the high, double-curvature, thin-concrete arch dam; the extensive model-testing program; the reliable source of the municipal water supply; and the operation for peaking power with reregulating dam and pool.

GENERAL DESCRIPTION

The Karadj Dam is located on the Karadj River, approximately 25 miles from the city of Tehran, Iran (Fig. 1). The project is the culmination of many years of effort to assure an adequate and reliable supply of municipal and industrial water for the rapidly growing capital city of Tehran. A secondary benefit of the project is hydroelectric peaking capacity to augment the electrical-power resources of the Tehran area.

The project is of major proportions and has several features of special interest. At the time of completion, the double-curvature arch dam will be the tenth highest dam in the world and the highest dam in the entire Middle East area. The dam rises 178 m (584 ft) above the lowest point in the foundation. The gated chute spillway on the right abutment is designed to pass a 52,000-cfs design flood. The flow is contained by training walls down to the terminal deflector; at this point, 50 ft above the river, the flow is turned away from the abutment and lands in the river well downstream of the dam and powerhouse.

Note.—Discussion open until January 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 4, August, 1960.

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The aboveground powerhouse provides space for three generating units of 111,000 kva total capacity. Two units will be installed immediately, and the third unit is scheduled for installation at a later date as required by the load growth.

HISTORY OF THE PROJECT

For many years, Tehran was a small village in rural Iran, and was relatively unimportant when compared with the ancient centers of commerce and culture–Isfahan, Shiraz, and Tabris. The energy requirements of the village were nonexistent and the water supply was adequately handled by a series of long, hand-excavated tunnels called "quanats." These tunnels occasionally extended for distances of 10 miles or more under the alluvial soil to reach an aquifer in the

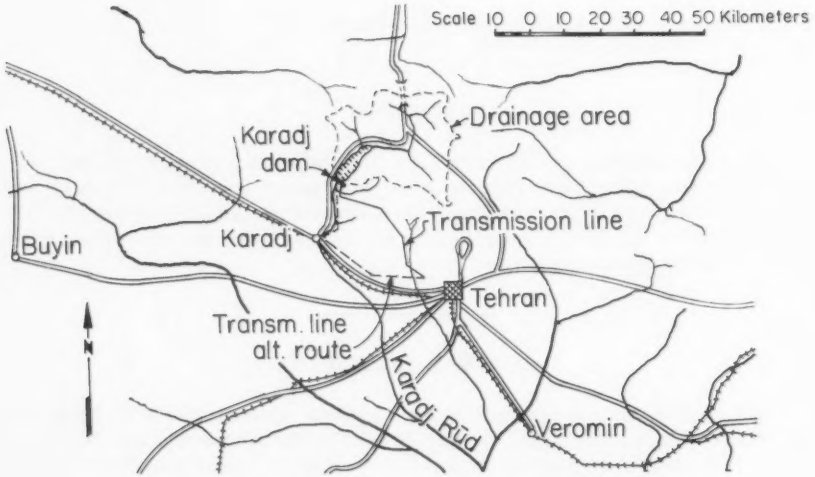

FIG. 1.—MAP OF TEHRAN AREA

foothills. Water flowed through these tunnels by gravity to a point on the plain such as Tehran or to agricultural areas for irrigation.

In the 19th century, the capital of Iran was moved to Tehran and a rapid and continuing growth of population commenced. Tehran is now (1960) one of the largest cities in the world not located on a body of water or a river. The local annual rainfall of only 10 in. was insufficient to support the needs of either city water supply or irrigation needs, and, therefore, as early as 1937, consideration was given to the development of a storage dam at Karadj. The earliest studies of this site were made under the direction of German engineers.

After World War II, the increased growth of the city led to a critical situation in water supply and electrical energy. Studies of the Karadj damsite were resumed and various plans were prepared. One study was made by a French firm in collaboration with Albert Caquot and another, in 1954, by a study team of the

Bureau of Reclamation, United States Department of the Interior. Both of these suggested developments called for a high arch dam together with a hydroelectric power plant located on the diorite sill formation, which forms the site of the present Karadj Dam. Late in 1955, a modern water-treatment plant and municipal water-distribution system went into operation. Steps were then undertaken to implement the Karadj Project as a reliable source of water.

The project will be financed with funds available from oil royalties, and will thus utilize a depleting energy source to construct a permanent water conservation and energy resource in the form of a 115,600,000 cu m (94,000 acre-ft) annual storage reservoir and a hydroelectric capacity of 149,000,000 kwhr per yr.

PURPOSE OF PROJECT

The primary purpose of the project is to provide a dependable industrial and domestic water supply for the rapidly growing city of Tehran. The development of water supply at Karadj will also provide some benefits to the flood control of the area. It was found economical in connection with the development of water supply to install a hydroelectric plant to provide peaking capability for the Tehran electrical system.

The total annual charges, using an assumed interest rate of 4 1/2%, are estimated to be equivalent to $3,700,000. Just over 50% of this will be recovered from electric revenues and the remainder will be charged against water supply. When fully utilized, this will provide water at $0.01 per cu m and this is deemed to be reasonable under the circumstances. Benefits to result from irrigation water supply and flood control have not been estimated for the purpose of justifying the project. In arriving at the foregoing annual charges, the project equipment is assumed to be fully depreciated over a 30-yr period, the power plant structure over a 50-yr period, and the main dam and spillway over a 100-yr period.

SELECTION OF TYPE OF DAM

During the course of the studies made of the Karadj Project, consideration was given to various heights and structural types of dam. These studies soon centered around reservoir elevations between 1750 m and 1760 m and an arch dam or a rockfill dam. A report submitted on August 15, 1956 made a final comparison of arch dam and rockfill dam types for a range from El. 1735 m to El. 1780 m. An international board of consultants consisting of Jean Bellier (France), Robert Fitt (England), and I. C. Steele (F. ASCE, United States) was assembled to study the report and to recommend a final height and type of dam. The consultants agreed with the recommendation set forth in the report; that is, a double-curvature arch dam with crest at El. 1768 m.

The topography of the site was almost ideally suited to an arch dam. On the other hand, the steep canyon walls presented a problem in the design of the core of a rockfill dam of this magnitude that would adequately provide a permanent seal at the contact with the abutments. In addition, there was no proximate source of core material, the nearest source of sufficient volume being about 20 miles downstream. The economic studies indicated that there was no advantage in the rockfill dam layout.

DESCRIPTION OF PROJECT

The Karadj Project (Fig. 2) consists of the main arch dam, a gated concrete chute spillway, reservoir outlet works, power plant, transmission lines, a downstream reregulating dam, and 9 miles of relocated highway. The significant project data is shown in Table 1.

From the deepest point in the excavation to the roadway, the dam will be 178 m high, and the arc length measured at the roadway is 390 m. The main dam (Fig. 3) is of the double-curvature type, with horizontal circular arches of approximately constant central angles. The radii of the arch center lines vary from a minimum of 74 m at El. 1612 m to a maximum of 201 m at El. 1770 m, the roadway

TABLE 1.—SIGNIFICANT PROJECT DATA

Hydrology	
Drainage area, in sq km	764
Average annual runoff, in 10^6 cu m	472
Maximum annual runoff, in 10^6 cu m	918
Minimum annual runoff, in 10^6 cu m	272
Flood of Record, in cu m per sec	350
Maximum probable flood (spillway design flood), in cu m per sec	1,450
Reservoir	
Main dam, live storage capacity, in 10^6 cu m	172
Reregulating dam, live storage capacity, in 10^6 cu m	0.6
Additional annual water supply available because of storage, in 10^6 cu m	115.6
Main Dam	
Maximum height, in m	180
Length, in m	390
Volume of concrete, in cu m	720,000
Reregulating Dam	
Maximum height, in m	50
Length, in m	93
Volume of concrete, in cu m	35,000
Hydroelectric Plant	
Capacity, initial, in kw	80,000
Capacity, ultimate, in kw	120,000
Average annual energy delivered, in kwhr	149,000,000
Estimated Cost of Project	
Total US dollar equivalent	50,000,000

level. The arch thickness is 32 m at El. 1612 m, varying to 7.85 m at the roadway.

The dam is analysed as a symmetrical structure with the base at El. 1600 m. Below this elevation, and on the right abutment, in the lower portion, plugs of massive concrete are constructed. These plugs are assumed to function as foundation rock, and are not analysed as a portion of the arch structure.

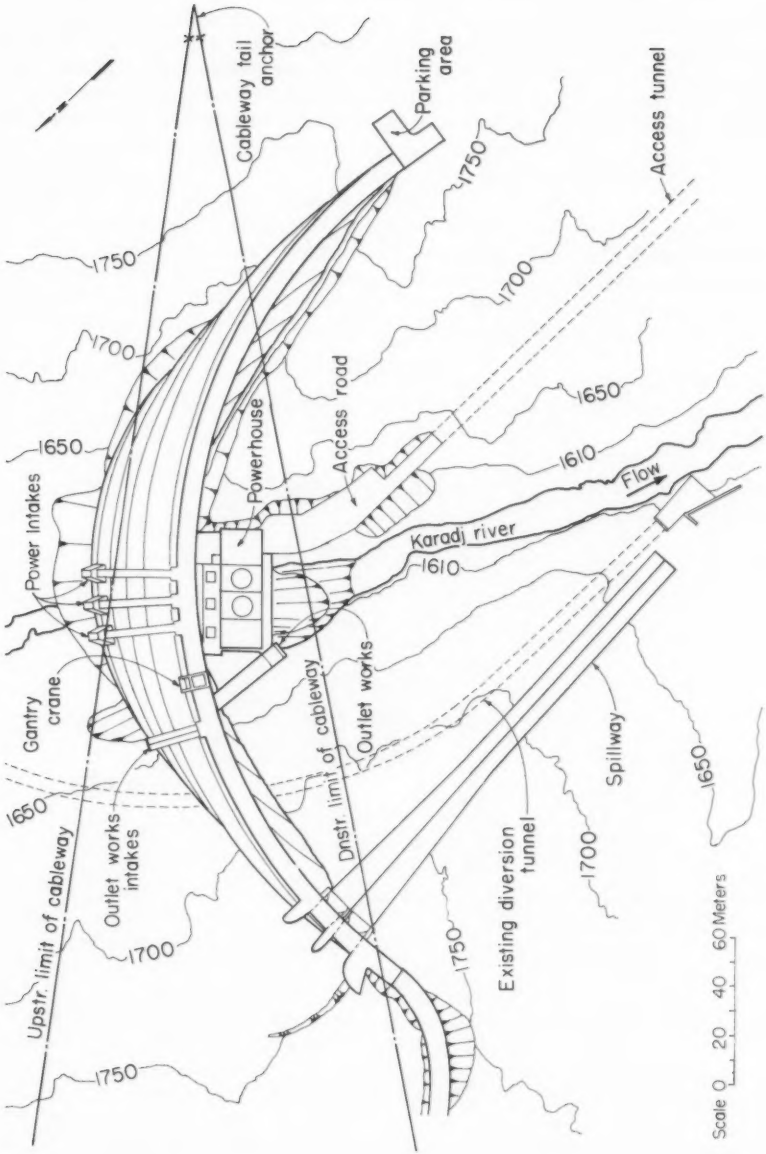

FIG. 2.—PLAN OF MAIN DAM, SPILLWAY, AND POWERHOUSE

The dam was initially proportioned and shaped to fit the limited area of sound foundation and abutment rock that exists in the canyon. This structure was then analysed by the crown cantilever method to determine the general magnitude of stresses and to find if any major revisions were necessary. After minor adjustments, the stress analyses were continued. The analyses of the arch dam confirmed that the double-curvature shape permits very efficient structural

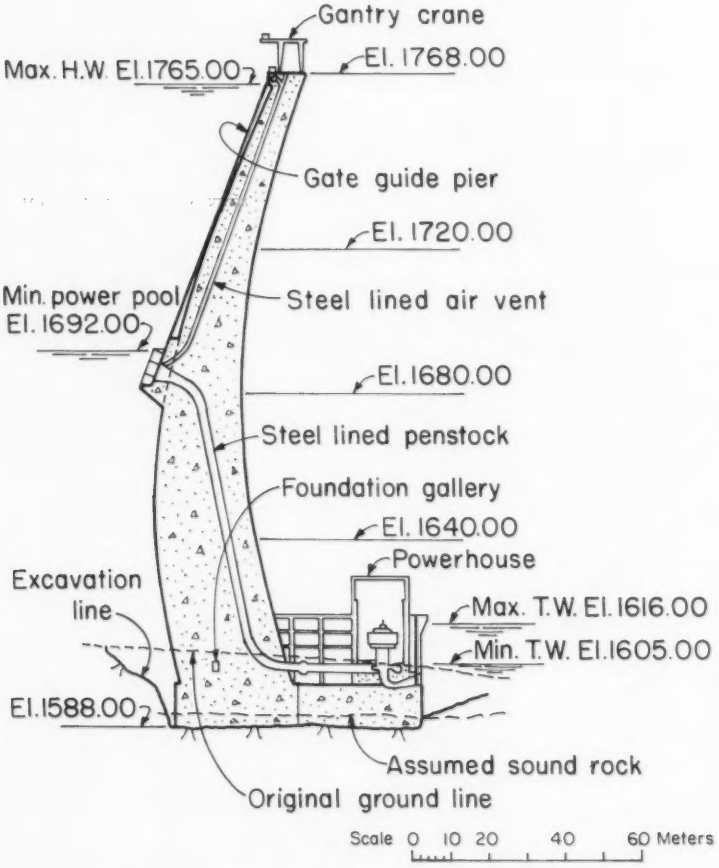

FIG. 3.—SECTION OF MAIN DAM

utilization of the concrete in the main dam. With careful planning of construction operations (such as schedule of concrete pouring, joint spacing, cooling and contraction joint grouting), the double-curvature design provides substantial savings in concrete volume as compared to other arch-dam shapes and contains approximately 60% less concrete than a gravity dam type for the same site. The next analyses made were the three and five cantilever analyses with

radial adjustments. The computed stresses were within the range of allowable stresses; that is, 150 psi tension and 1,000 psi compression.

The following step in the arch dam design was to arrange for a complete structural model-test program. The combination of the structural tests with electric strain gages on a series of plaster models together with the algebraic trial-load analysis performed with the IBM-704 electronic computer served to confirm the suitability of the original dam structure layout, shape and outline. Furthermore, certain minor modifications to dam shape were suggested from the results of the testing program.

The model studies also answered two economically important design questions: First, thrust blocks would not be necessary on either abutment and secondly, the opening in the arch dam to provide for the spillway crest piers and gates did not materially affect the stresses in the dam nor the abutment loads and, therefore, a thrust beam would not be required across the top of this opening.

The model-study program was planned to include tests that would incorporate the effect of the differential in the modulus of elasticity of the foundation rock

FIG. 4.—MODEL OF KARADJ DAM IN PLASTER FOR STRUCTURAL LOADING TESTS

relative to the concrete in the arch dam. However, the field in-situ rock modulus tests performed in galleries in the foundation revealed that the foundation modulus of elasticity was quite close to that of the concrete. Foundation moduli measurements ranged generally from 2×10^6 psi to 2.5×10^6 psi, while the value for concrete in the main dam is 3×10^6 psi. Therefore, the tests were not required and the material used to construct both the model of the dam and foundation was homogeneous plaster. (Fig. 4)

The arch-dam concrete will be cooled in two stages by circulating water through embedded pipes. The first-stage cooling will reduce the temperature rise developed by the heat of hydration. The second-stage cooling will reduce the temperature of the mass concrete about 5° F below the average mean temperature of 54° F and will be accomplished prior to grouting the contraction joints of the dam. Either stage of cooling may utilize river water without artificial refrigeration. However, for the frequent occasions when the river water is too warm or when more rigid control of cooling-water temperatures is required, a 800-ton refrigerating plant is provided. The second-stage cooling

operation will be performed under closed-system circulation to reduce the head required for the pumps and also reduce the volume of water treated for removal of silt or suspended solids.

The mass concrete for the arch dam is designed to develop 4,000 psi at 1 yr. This concrete strength has been obtained in a mix with a cement content of 3 1/2 bags per cu yd. This content may be reduced near the crest of the dam where stresses are lower. Also additional experience as concreting operations progress may permit a reduction in cement content if actual strengths exceed the design requirements.

The foundation of the dam is a massive diorite sill. The presence of joints and fractures requires a careful grouting program to assure that the rock will act as a homogeneous mass. To assist in this program, and to permit grouting to be done without interference with the schedule of concrete placement, a drainage and grouting gallery is provided near the foundation throughout the length of the dam.

Two basic considerations were given to the underdrainage system. First, the uplift pressures are of minor significance to the design of an arch dam as compared to a gravity dam. Secondly, in the vicinity of the powerhouse relief of uplift pressure by adequate provision of drain holes and foundation drains was desirable to assure the stability of the powerhouse. Uplift pressures were assumed to act over 100% of the area of the section with pressure diminishing on a straight-line variation from headwater to atmosphere or tailwater. One drain hole for each block of the arch dam was provided to check the effectiveness of the seepage cut-off grout curtain. In the blocks immediately upstream of the powerhouse drain holes were spaced 10 ft on centers to intercept all seepage and relieve uplift pressures. Drainage of the foundation will be through the gallery.

Instrumentation.— The program of instrumentation of the main dam includes the locating during construction of 161 thermometers, 58 joint meters, 536 strain meters, 16 foundation deformation meters, and 21 embedded targets on the downstream face of the dam. A recording and analysis program is planned to provide a correlation between the prototype function, the model tests, and the trial-load analyses.

MAIN SPILLWAY

The spillway (Fig. 5) is located on the right abutment (Figs. 6 and 7). Control is by two 32.8 ft tainter gates with the discharge carried in a chute. At a point well downstream of the dam and powerhouse, floods will be discharged into the river by a terminal deflector structure at the end of the chute.

The gates will normally be operated by electrically driven hoists equipped for either simultaneous or individual operation. Local control at the spillway hoists, and alternative remote control from the powerhouse are provided. In case of power failure, the gates may be operated by manual-driven connection at the hoists.

Hoists are located on the piers downstream from the roadway. The chain from the hoist drum is connected to the downstream side of the gate for ease of inspection and maintenance.

Hydraulic model tests were performed by the University of Toulouse to determine the optimum approach shape, chute cross section, chute transitions

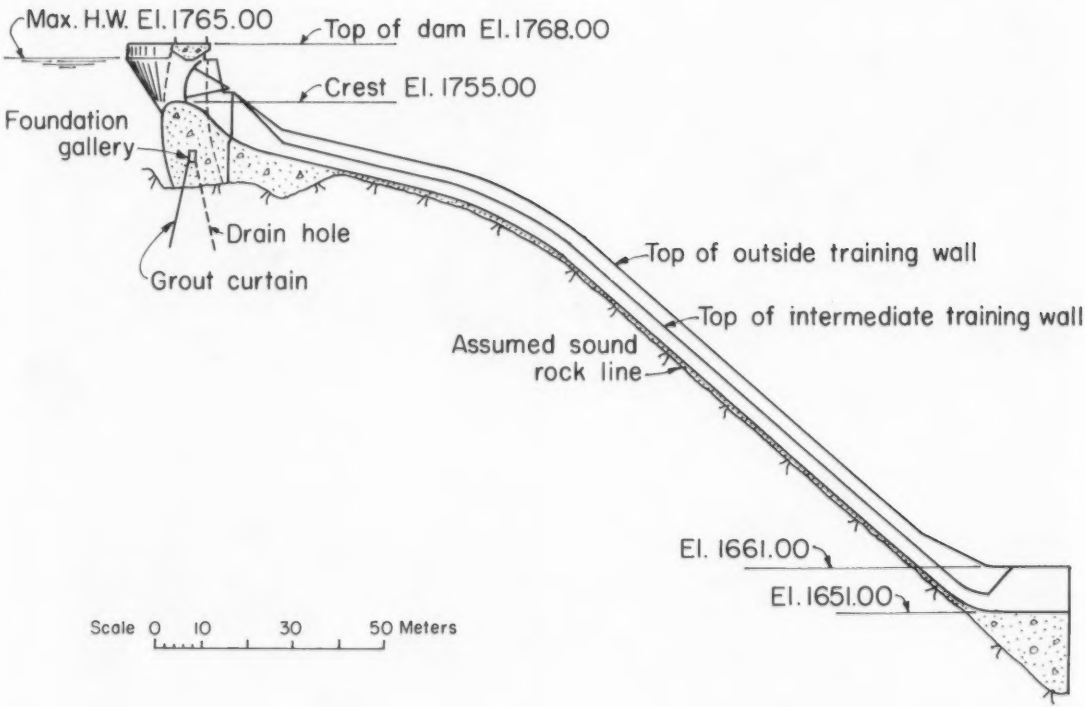

FIG. 5.—SECTION OF MAIN DAM SPILLWAY

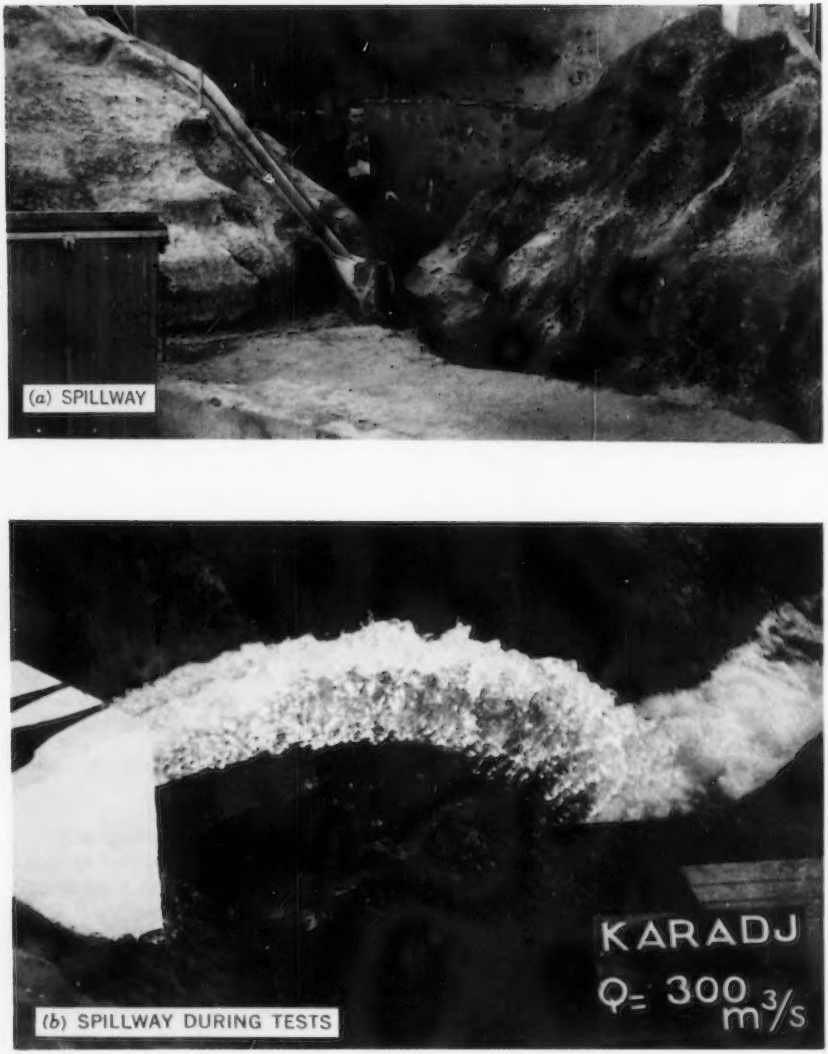

FIG. 6(a,b).—HYDRAULIC MODEL

and superelevations, and to satisfy discharge conditions below the end of the chute (Fig. 6(a).

Two phases of the hydraulic model work were unusual. Due to the profile of the foundation, water in the chute reached a high velocity at a point upstream from the transition section. This required a special study to reduce the height of the standing waves by carefully determining the curvature of the transition and by superelevation of the chute floor in this area.

Due to flow characteristics of the river, in some years the spillway may operate frequently and under varying flows. To reduce the use of the river outlet valves the spillway will be used to waste all discharges above 30 cu m per

FIG. 7.—VIEW LOOKING UPSTREAM SHOWING EXPOSED ROCK FOUNDATION FOR CHUTE SPILLWAY (JULY 12, 1959)

sec. Thus it was necessary to design a terminal deflector which would operate through a wide range of flows and discharge the water out into the river to prevent erosion immediately below the structure. After many trials in the model a shape was found which would operate under low flows, but which would not "clog-up" under high flows (Fig. 6(b).

The foundation rock is very sound, but due to the joint system of the mass surface is quite irregular after excavation. Seamy or jointed rock will be grout-

ed prior to placing concrete. Low spots or areas will be filled with dental concrete to form a rough foundation grade; then anchor bars will be drilled and grouted into rock in certain areas. Drains will be formed on this rough surface to prevent uplift from raising the slab should dynamic pressure develop due to the high-velocity flow.

The slab and walls will be reinforced, with special care taken in the design of shrinkage reinforcement. A minimum number of joints are used and these are provided with water stops and under drains.

During construction care will be taken in the interior surface finish of the chute and at joints to reduce erosion and cavitation.

A center wall throughout the length of the structure is provided to control the higher flows when the two gates will be operated simultaneously, and during low flows will permit either gate to be operated separately. This also has the advantage, in case of damage to the chute, that repairs may be made on one side while the other is available for passing lower discharges.

OUTLET WORKS

Outlet works were provided through the right side of the dam to permit controlled release from the reservoir for downstream use in case of emergency. The capacity of each of the two outlets is sufficient to pass the required flow at all reservoir levels. These outlets also control the river flow during critical periods of reservoir filling during construction, prior to powerhouse completion.

Two outlets are provided. These consist of gated intakes, with trash-rack structures, steel-lined conduits 730 cm in diameter through the dam, with hollow-jet-dispersion type regulating valves, 76 cm in diameter at the outlet end.

The intake gate will be of the wheeled type for emergency closure operated by a fixed hoist near the roadway level. Normally this gate will be in position over the intake and the conduit will be dry. Filling of the conduit for operation may be accomplished either through the cross connection with the penstocks, or by cracking of the intake gate. Air vents are provided downstream from the gate.

The center lines of the intakes are at El. 1660 m which is 5 m above the maximum level of sediment deposits expected after 100 yr.

The regulating-valve structure is located to the right of and is monolithic with the powerhouse. Discharge of the valves is at an angle to the transverse center line of the powerhouse and is directed downstream into a bend in the river.

For architectural appearance the downstream edge of the valve structure is on the same line as the draft-tube deck. The roof of the valve structure will be at the same level as the draft-tube deck, with valves and controls located below. Maintenance will be by means of removable hatch in the draft-tube deck.

POWERHOUSE

The powerhouse (Fig. 8) is of the conventional indoor type located just downstream of the dam. Space for three units is provided; two are to be installed initially.

The turbines are vertical shaft, Francis type, rated 49,700 hp at full gate and 456 ft net head, 300 rpm. The spiral cases are of welded steel.

The generators are of the umbrella type, rated at 40,000 kw, 13.8 kv, and 0.95 power factor. They are totally enclosed and furnished with surface air coolers to provide a self-ventilating system.

Station electrical service will be 230/400 volts, four wire, 50 cycle. When the main units are operating, service power comes from either of two 500 kva transformers fed from generator main leads. During off peak (non-operating hours) station service and operator's village supply will be from two 1,000 kw diesel-driven sets located in a separate building 300 m downstream from the reregulating dam on the left bank of the river.

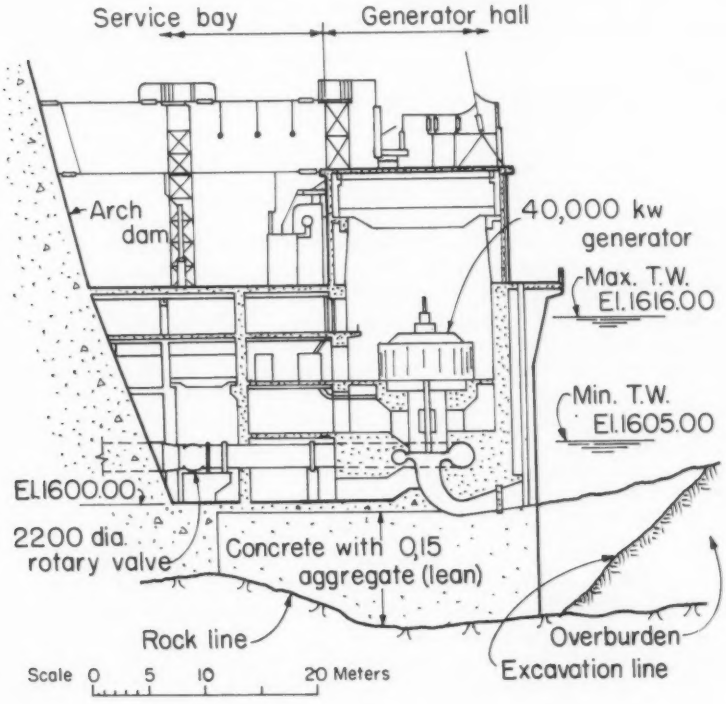

FIG. 8.—SECTION OF POWERHOUSE

The powerhouse was held as close to the dam as practical for economic reasons, but due to the double curvature of the dam, and the fact that tailwater required a relatively high draft-tube deck, an area between the dam and the powerhouse at El. 1615 m resulted. Advantage of this area is taken by placing the transformers and a portion of the switchgear here. The switchgear will be located partially in this area and partially on the roof of the powerhouse. The lighter weight equipment will be located on the roof.

Rooms below this platform area will be provided for control, offices, storage, switchgear, and auxiliaries.

Three 2.6-m diameter steel-lined penstocks are provided through the dam. Individual intake structures for each of the penstocks are provided and a wheeled gate protects each opening for emergency closure and closure for inspection of the penstocks. The intake structures also contain the trash racks. The gates will be raised to the crest by individual fixed chain hoists. A gantry crane is provided on the crest of the dam for removal of the gates for inspection and maintenance.

The embedded portion of the penstocks is designed as a water-tight membrane. Provision for drainage is provided to relieve external pressures, and resulting stresses when the penstocks are unwatered. The penstocks are of welded construction throughout their length. Inspection of welds will be by radiographic and ultrasonic methods. All longitudinal welds are to be inspected by

TABLE 2.—PRINCIPAL ITEMS OF EQUIPMENT AND SOURCES

Item (1)	Description (2)	Country from which Supplied (3)
Generators	Two 42,100 kva	Japan
Transformers	Two 46,200 kva	Japan
Turbines	Two 49,700 hp	Scotland
Spherical valves	Two 2.2-m diameter	France
Cranes		
Powerhouse	140 metric tons	West Germany
Intake gantry	20 metric tons	West Germany
Valve chamber	45 metric tons	West Germany
Gates		
Spillway	10 m by 10 m	West Germany
Intake, power	2.5 m by 5.0 m	West Germany
Intake, outlet	1.3 m by 2.6 m	West Germany
Draft tube	2.8 m by 2.5 m	West Germany
Outlet valves	Two at 760 mm	Austria
Reregulating service	Two at 1.52 m by 1.22 m	Japan
Penstocks		Austria
Elevator	One at 1800 kg	Switzerland
Transmission line		
Towers and conductors		Italy
Hardware		Japan
Diesel generator sets		U. S. A.

radiograph for quality. Design working stress is 16,000 psi, and a joint efficiency of 90% is assumed.

Service valves will be provided immediately upstream of each unit and because of operation of the turbines as peaking units these valves will be operated daily to relieve the headwater pressure on the wicket gates.

The service valves will be 2.2-m diameter spherical valves.

An unusual feature of the powerhouse is the depth to rock in the central portion of the river, requiring concrete backfill under the draft tubes. Advantage was taken of this feature by locating the temporary diversion conduits in this mass. The conduits are steel lined and will be backfilled and grouted when this phase of the diversion is complete.

The principal equipment and the countries from which they were obtained are presented in Table 2. The many sources of supply points up the coordination aspect of the engineering.

REREGULATING DAM

The reregulating dam is located 4,000 ft downstream from the power plant (Figs. 9 and 10). It is an arch gravity concrete dam with a centrally located gated spillway. The two spillway gates are the same size as those used in the main dam spillway. Layout of the gates and hoist and operation of the hoists will be similar to the gates on the spillway of the main dam.

The outlet works are located in the dam to one side of the crest structure and consist of two 4.0 ft by 5.0 ft sluice gates. Either sluice gate will pass the maximum required discharge for downstream water demand. The two sluice gates will discharge into tailwater. Individual intake portals will give greater flexibility of operation and permit maintenance and repairs on one sluice gate while the other is in use. Air vents downstream from the sluice gates are provided. Diversion during construction will be provided by a temporary concrete-lined tunnel through the right abutment. This tunnel will be plugged and abandoned after construction.

TRANSMISSION LINE

The power developed at Karadj will be transmitted over a single-tower, double-circuit transmission line to a receiving substation located in the suburbs of Tehran. The transmission line will be 33 miles long and traverses extremely rugged terrain for the major portion of the distance. Transmission is at 132 kv at 50 cycles. The alienment roughly parallels the Karadj river valley to the plains downstream and then follows the foothills until a point near Tehran. This alienment was found to be the most economical and avoided traversing the costly property of the plains between the Karadj village and Tehran. Initially only one circuit will be installed on the towers, the second circuit will be installed at the time the third unit is erected in the powerhouse.

CONSTRUCTION

The Karadj project is scheduled for completion in mid-1961. The rugged topography at the damsite (Figs. 11 and 12) presented problems in the setting of the construction plant. Since very few level areas existed, most of the construction-plant facilities required extensive excavation. Both fine and coarse aggregate must be manufactured and for this reason the limited storage area available is a critical point in the construction program.

Concrete placing schedule in the main dam calls for an average of 10,500 cu m per week. The concrete is handled from the batch plant in 8-cu-yd buckets on rail cars to the cableway. There are two traveling head towers on the right abutment and they provide service to a major portion of the dam and powerhouse. Maximum permissible main-dam-concrete lift is 2 m. A portion of the concrete in the dam on the upper left abutment will be placed by a stiffleg derrick. Also the spillway chute and bucket concrete will necessitate pumpcrete operations for placement.

During construction the Karadj river is to be diverted first through a 4.9-m diameter concrete-lined tunnel in the right abutment. After the arch-dam concrete has progressed sufficiently high above the canyon floor diversion will be transferred through three conduits embedded in the base of the dam and extending through the powerhouse substructure backfill concrete. Two conduits will

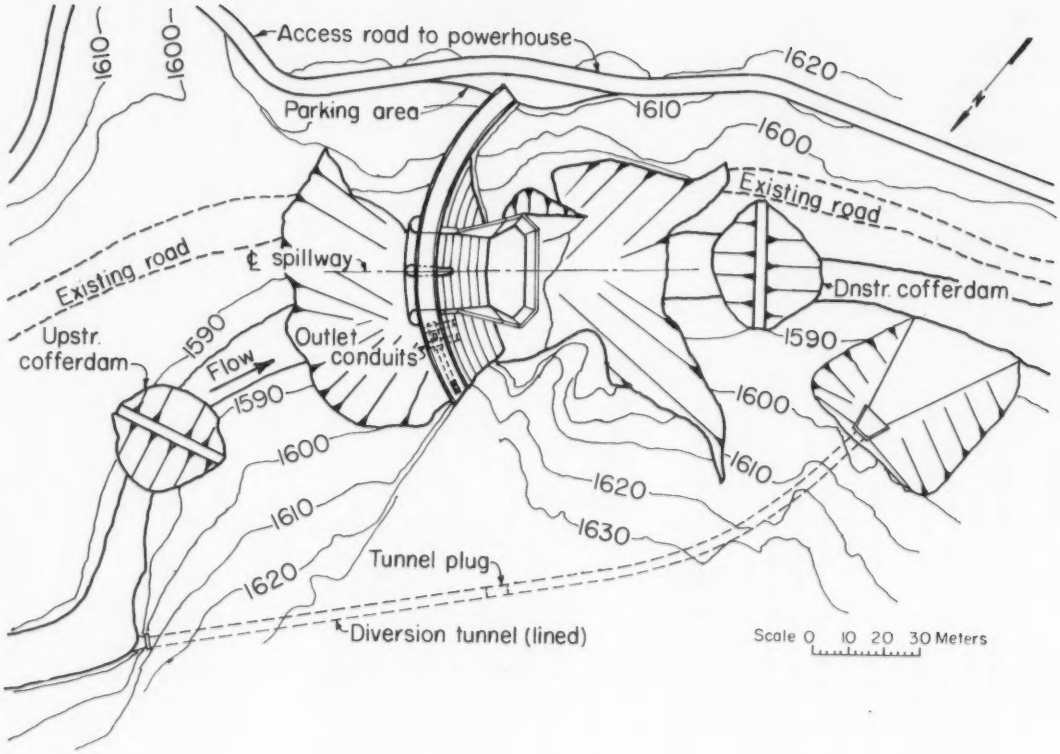

FIG. 9.—PLAN OF REREGULATING DAM

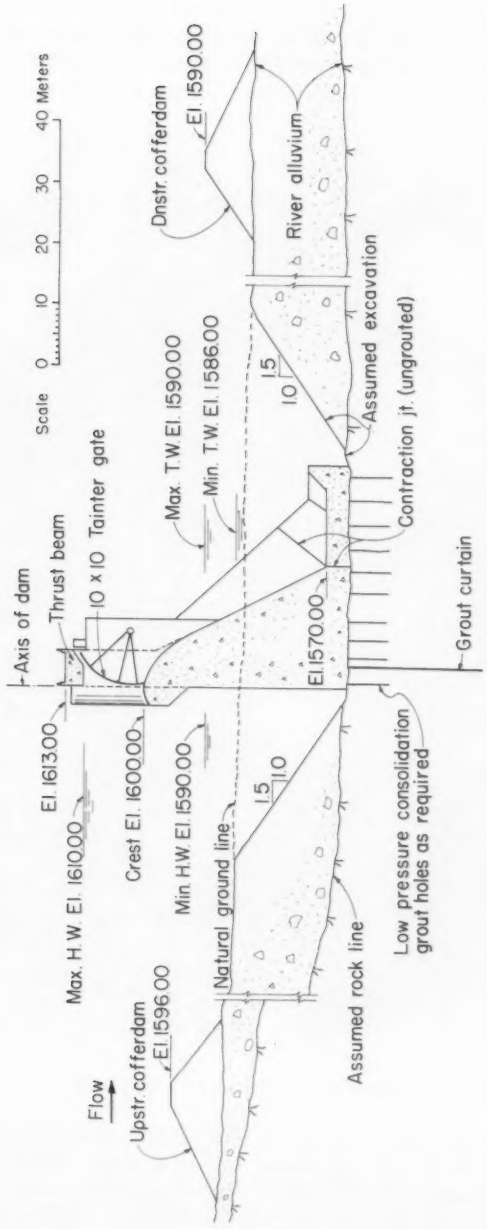

FIG. 10.—SECTION OF REREGULATING DAM

be provided with spherical valves that will be removed subsequently and installed in the penstock system. The third conduit will be provided with a 1.5-m diameter butterfly valve and will permit regulation of flows downstream. The magnitude of downstream releases is established by (1) the downstream demand for water supply and irrigation and (2) regulation of the reservoir elevations during construction. The spherical valves can only discharge in the full open position and provide the necessary releases for flood control.

FIG. 11.—GENERAL VIEW OF EXCAVATION OF LEFT ABUTMENT (JUNE 1, 1959)

The heaviest pieces to be shipped are the three-phase main power transformers at 47 metric tons each. Shipping dimensions are 3.54 m by 2.8 m by 4.2 m.

The steep slopes in the Karadj canyon made excavation difficult and dangerous. The many systems of jointing in the foundation rock resulted in isolated blocks that had to be removed or anchored to the abutment to prevent loosening and falling on the workers below. Even under these adverse conditions construction is proceeding rapidly and will follow the original schedule (established in 1956) that calls for completion in mid-1961.

CONCLUSIONS

A large, multiple-purpose dam project is being constructed to provide municipal water supply and electric peaking power capacity for Tehran, Iran. Water

FIG. 12.—VIEW OF MAIN DAM CONSTRUCTION OPERATIONS (JULY 12, 1959)

supply and power are the sources of project revenue, with flood control and irrigation as additional benefits.

The dam structure is of the double curvature arch type. During design, special efforts were exercised to test and analyze the foundation, and extensive electronic computor analyses and model tests of the arch structure were conducted. Dam and power plant construction are complicated by problems of steep site topography, concrete cooling and joint grouting requirements, foundation

grouting and under drainage problems, and extensive main highway relocation works.

The project involves problems of coordination involving use of local cement and aggregate by local labor in a complicated major arch dam structure constructed by an American general contractor. Additional coordination is required by employment of permanent project equipment from several European countries and Japan.

Design, coordination, and construction are proceeding on schedule and the Karadj Dam Project will go into service during 1961.

ACKNOWLEDGMENTS

The Karadj Hydroelectric Project will be owned and operated by the Karadj Dam Authority, which is a division of the Plan Organization of the Government of Iran. Harza Engineering Company International was engaged to design and supervise the project with Morrison-Knudson Company International as the constructors.

The laboratory selected to perform the model-test program was Electricite de France located at Chatou near Paris. The model studies and later the electronic computer program of the trial-load type was administered by the Compagnie Francaise d'Entreprise of Paris and directly supervised by engineers of the Harza Engineering Company International.

The responsible representatives of the Iranian Government for the project include S. Asfia, Deputy Director, Plan Organization and Khalil K. Taleghani, M. ASCE, formerly Director, Karadj Dam Authority. H. E. Buckert is Resident Manager for Morrison-Knudson International Construction, Inc.; and for Harza Engineering Company Bruce M. Johnson, M. ASCE, is Project Manager in Chicago while Robert E. Brown, AM. ASCE, is Resident Engineer.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 4, August, 1960.

BUREAU OF RECLAMATION PRACTICES FOR CONTROL OF CRACKING IN ARCH DAMS[a]

Closure by Charles L. Townsend

CHARLES L. TOWNSEND,[1] F. ASCE.—The writer would like to reiterate the thought that the type and degree of temperature control adopted for any large concrete dam is dependent upon the size and type of structure and the climatic conditions at the damsite. For example, the Glen Canyon Dam which is presently under construction on the Colorado River in northern Arizona is an arch dam. In order for the dam to carry its load to the abutments by arch action the contraction joints must be grouted. An embedded pipe cooling system must be used, therefore, to accomplish the required cooling prior to contraction joint grouting. Further, the size of the construction block and the climatic conditions existing at Glen Canyon are such that precooling measures were deemed necessary. The maximum temperature of the concrete as it is placed in the forms was limited to 50° F so that the subsequent temperature drop from the maximum temperature, attained 4 to 5 days after placement, to the grouting temperature would not exceed 35° F.

The writer would like to present the experiences at Hungry Horse and Monticello Dams. Those are the most recently completed arch dams designed by the United States Bureau of Reclamation, Dept. of Interior (USBR).

Hungry Horse Dam.—Hungry Horse Dam is located on the South Fork of the Flathead River, about 9 miles southeast of Columbia Falls, Montana. The dam is a variable thickness, concrete arch dam approximately 2,100 ft long at the crest. It rises to a maximum height of 546 ft above the foundation and has a maximum base width of approximately 320 ft. The top width is 39 ft and accommodates a 30-ft roadway with sidewalks on each side. Mass concrete in the dam totaled roughly 2,950,000 cu yd.

Crack prevention measures at Hungry Horse Dam consisted primarily of a reduction in the length of block and the use of embedded pipe cooling systems. Because of the dam's thickness, a longitudinal joint was placed in each block so that the largest individual block would be 186 ft in length. With blocks this long, it was still desirable to hold the temperature rise to a minimum so that there would be no trouble from the subsequent temperature drop to the grouting temperature. Embedded cooling systems were therefore operated to reduce the maximum temperature of the concrete as a crack prevention measure.

Since these systems were already required to cool the concrete to its grouting temperature, so that the structure could be grouted and put into service at the earliest practicable date, this additional use was accomplished at very little cost. The effectiveness of the cooling systems in preventing structural

[a] August, 1959, by Charles L. Townsend.

[1] Engr., Concrete Dams Sect., Div. of Design, Commr's. Office, Bur. of Reclamation, Denver, Colo.

cracking was proven by the results. The average temperature rise, as found by resistance-type thermometers, was 18.3° F. This rise, combined with placing temperatures as high as 65° F resulted in maximum concrete temperatures of 85° F. Subsequent temperature drops, to the grouting temperature, as great as 47° F were forced on the concrete by the cooling systems with no cracking.

Only one crack occurred that could be called a prominent structural crack, and this occurred in concrete placed in the fall of 1949. This crack occurred in Block 16B when the top of the block was at elevation 3050, only about 20 ft off the foundation, and was left exposed over the 1949-50 winter season. The crack extended completely across the block and opened 3/64 in. Treatment of this crack consisted of placing a mat of reinforcing steel over the crack to prevent its extension into higher lifts.

Minor cracking did occur in four general areas at Hungry Horse Dam, even though crack prevention and temperature control measures were employed. Cracking in all four of these areas took place within the first month or so after placement of the concrete involved. These four general areas can be summarized as occurring under the following conditions or at the following locations:

1. At the upstream and downstream faces of the dam, where construction joints were left exposed for an extended period of time
2. Across the tops of blocks that were left exposed for longer than normal periods
3. In galleries and on contraction joint faces
4. At the top of the dam

Cracking of Horizontal Construction Joints Left Exposed for Extended Periods of Time.—The program of treatment previously described in this paper for horizontal construction joints left exposed over extended periods was not in effect during the construction of Hungry Horse Dam. Although experienced on other projects to a smaller degree, the parting cracks along the horizontal construction joints left exposed over winter periods were noticeable on the upstream and downstream faces at Hungry Horse Dam.

Virtually every joint left exposed during the 1950-51 winter and the 1951-52 winter opened at the faces of the dam and leaked water or grout. These cracks caused trouble during the contraction joint grouting because they extended behind the grout seals placed on the vertical transverse joints. Grout injected into the vertical joint passed the seals and spread out into the horizontal crack before coming to the surface. These cracks then had to be calked before contraction joint grouting could be completed. In addition to being a point of structural weakness, extending into the mass as much as 12 ft, these cracks are quite often the beginning of a progressive deterioration and spalling due to the freezing and thawing of water that accumulates in the crack during later years.

Cracking Across Blocks Placed at Irregular Intervals.—Delays in placement caused minor cracking across a number of blocks where there was a delay in the regular placement schedule. As with most cracking in concrete, these cracks were very erratic. Most of these cracks were hairline cracks ending a few feet from the surface, although some did extend through 2 or 3 lifts before stopping. The most severe of these cracks occurred in those few lifts of concrete placed in the fall of 1949.

In order to get out of the bottom of his excavation and in order to have room to place the next spring, the contractor started placement of concrete in the

dam in September and stopped early in November. During this time, he placed from 5 to 10 lifts in each of about 8 blocks. Being close to the foundation where restraint was high, and being subjected to rapidly dropping air temperatures, which caused concrete temperatures to drop rapidly after the maximum temperature was reached, these blocks developed cracks that were generally from 1/64 to 1/32 in. in width. In general, these cracks occurred at the approximate third points of the block, and several cracks completely crossed the block.

This same type of crack occurred during 1950, in the central portion of the dam and in concrete which was as much as 150 ft off the foundation. This time, however, the cracking occurred during the summer months when placing temperatures were about 60° to 65° F and night-time exposure temperatures were around 30° F. These cracks almost always occurred in those blocks where there was a delay in placement of 7 or more days. The size of the block apparently did not have any effect because the cracks, usually hairline, would quite often appear at approximate 25-ft intervals in both directions. High blocks were particularly susceptible to this type of cracking.

Cracking in Galleries and on Contraction Joint Faces.—Minor cracking on contraction joint faces occurred during the first year of placement. In most cases, these cracks were hairline and occurred in the recessed part of the contraction joint keys. Hairline cracks were also quite extensive in the galleries, particularly in concrete placed during the 1950 season. These cracks usually appeared at the top of the gallery arch. Only those near a contraction joint leaked water.

Cracking at Top of Dam.—At the top of the dam, a number of minor cracks occurred in the top 5 ft of concrete. The first blocks in the dam were topped out in August, 1952, and a few hairline cracks were noticed shortly after that time. Initial cooling through the embedded cooling systems was immediately cut back to 5 days for these lifts. Falling air temperatures in September and October caused this relatively thin and newly placed concrete to undergo a rather rapid temperature drop. In almost every block, which, at this time, had a width of over twice the thickness, transverse cracks, running from upstream to downstream faces, occurred. Most of these cracks were hairline to 1/64 in. in width. A few, however, opened as wide as 1/32 in.

Monticello Dam.—Monticello Dam is a thin-arch concrete dam located on Putah Creek, about 30 miles west of Sacramento, California. The dam is approximately 304 ft in height, 1,000 ft long at the crest, and contains about 327,500 cu yd of mass concrete. Concrete placement started in August, 1955, and was completed in February, 1957. Crack prevention measures included placing a maximum value on the placing temperature and the use of embedded cooling systems, which were used to remove part of the heat of hydration immediately after placement and to cool the dam prior to contraction joint grouting. Summer temperatures at the site reached as high as 110° F, with average maximum air temperatures of around 95° F. With temperatures as high as these it was felt desirable to hold the placing temperature to 65° F so that the maximum temperature would not be too high. For the maximum size of block at Monticello Dam (55 ft wide by 80 ft long) it was felt desirable to hold the temperature drop to not more than 45° F near the base of the dam.

The results of these crack prevention measures is best typified by the March, 1957, report by project personnel which stated "There are no cracks in the concrete as observed from the upstream and downstream faces of the dam and from the gallery of the dam. There are three minor cracks in the

concrete of the roadway slab in Blocks 5, 7, and 8. There is no seepage or moisture showing in any of the contraction joint cracks in the gallery or at the downstream face of the dam."

Measured temperature rises in the concrete averaged close to 19.5° F. This was accomplished with 5-ft 6-in. nominal horizontal spacing of the cooling pipes on top of each 5-ft lift of concrete.

ERRATA

Journal of the Power Division

Proceedings of the American Society of Civil Engineers

April, 1960

p. 29. Please replace the paragraph beginning with the words "It is widely known that. . ." with the following:

It is widely known that the power potential of many great rivers, such as that of the Congo with 25 million kw at Leopoldville, cannot yet be harnessed because the minimum size of project has exceeded any possible use up to the present. It is not so widely appreciated, however, that many potential hydro projects remain too small to be attractive until the accessible market has grown to sizeable proportions on the basis of steam generated power. In some cases, a site, which might have contributed only a nominal dependable capacity to the load of the early 1920's, may have become capable of a substantial contribution to the present-day load. As explained in detail later, the larger load offers a shorter peaking period for minimum stream flow and thus increases potential firm capacity; a bigger load also can absorb more hydro energy at times of high stream flow. Outstanding enhancement is not, by any means, imparted in all cases of load growth, but occasionally a site which was quite properly rejected as too small to be of interest in an investigation long ago may be found to have attained very substantial capability. However, such increased hydro capability is of little importance to the power consumer unless it makes possible the production and sale of the hydro power at a cheaper rate than power from some other source such as a steam boiler.

p. 30, line 4 from the bottom. Delete the word "as" before the word "Competitive"

p. 31, line 14. Change "simple" to "simply"

p. 32, line 4. Delete hyphen between "minimum" and "water"

p. 32, 3rd line from bottom. Delete hyphen between "potential" and "power"

p. 34, 6th line from bottom. Remove comma after "potential"

p. 34, bottom line. Delete cms and substitute c.m.s. or m^3/s

p. 35, 8th line from bottom. Delete hyphen between "low" and "load"

p. 37, line 24. Delete "an" following "season of"

p. 37, 6th line from bottom. Change "incidental" to "incident"

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

VOLUME 85 (1959)

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184 (WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240 (PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

VOLUME 86 (1960)

JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502 (EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511 (EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520 (PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529 (HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580 (EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

c. Discussion of several papers, grouped by divisions.

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PART 2

AUGUST 1960 — 25 NO. PO 4
VOLUME 86 PART 2

Your attention is invited

NEWS OF THE POWER DIVISION OF ASCE

JOURNAL OF THE POWER DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

POWER DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

August, 1960

Appointment to Executive Committee

Mr. C. Russell Dole, Chief Civil Engineer of the Dayton Power & Light Company, Dayton, Ohio has been appointed to the Executive Committee to take office in October, 1960 in lieu of Mr. G. J. Vencill whose term of office will expire.

* * *

Mr. Marcel P. Aillery and Mr. R. N. Bergstrom represented the Power Division at the American Power Conference held in Chicago in April.

Mr. Louis G. Puls, Chief Design Engineer, Bureau of Reclamation has replaced Mr. Frank Cook on the Committee on Power Plant Planning and Design. Mr. Cook has rendered valuable service and we wish him an interesting retirement.

Rickey Medal Nomination

The Rickey Medal Committee has recommended, and the Executive Committee has unanimously approved, the award of the Rickey Medal to Mr. J. Barry Cooke "for his paper 'Hass Hydroelectric Power Project' and for technical contributions and work towards progress in the technology of rock fill dams." It is expected that the award will be made at the annual convention in October, 1960.

Passing of Mr. George R. Strandberg

The Power Division has lost a firm friend and former member and Chairman of the Executive Committee by the death of Mr. George R. Strandberg, retired Consulting Engineer of Stone and Webster Engineering Corporation. Mr. Strandberg was very active in Power Division work and contributed several papers to the Society.

Reno Meeting

The Power Division had three very successful sessions at the Summer Convention in Reno, Nevada, June 20-24. The United States Committee on

Note.—No. 1960-25 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 4, August, 1960.

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Large Dams (USCOLD) held 4 sessions at the same meeting, one of which was jointly sponsored by the Power Division.

The Committee on Power Project Planning and Design met in Reno and the following matters will be of general interest:

A Progress Report and Bibliography on Civil Engineering Features of Thermal Plants has been prepared and will be presented at the Boston meeting in October 1960.

The bibliography on Civil Engineering Features of Thermal Plants contains sections on site development, foundation considerations and treatment, water supply and circulating water systems, pedestal design, structural design of superstructure and structural features of stacks. A brief summary is given for each reference, and power engineers will find this a very useful index for reference.

A symposium on Pumped Storage Hydroelectric Plants is being planned for the Annual Meeting in October, 1961.

The various papers submitted in the Symposium on Rock Fill dams in Portland in June 1958, together with subsequent discussion, will be published as a special volume of "Transactions" and it is expected will be available early in 1961. It is expected that the cost will be approximately $5, but further information will be given in "Civil Engineering" later.

Mr. Von Gunten has sponsored a bibliography on "Fish Passage Facilities at Hydro Plants," with contributions from members of his committee and various Federal and State agencies. Copies of this bibliography may be obtained by Power Division members by writing to Mr. Glenn Von Gunten, 1509 Melrose St., Walla Walla, Washington.

Subjects for future activity of this Committee include a historical review and discussion of failures of structures and equipment as affecting structural planning, and the economics of lined versus unlined tunnels.

1960 Annual Meeting

At this year's annual meeting in Boston, October 10-14, the Power Division will hold 4 sessions of which two will be jointly sponsored by other divisions. The following papers will be presented, according to the tentative program:

MONDAY MORNING - OCTOBER 10, 1960:

Power Division and Construction Division

Operation Plowshare by G. W. Johnson - Lawrence Radiation Laboratory
Yankee Atomic Power Plant by H. T. Evans and L. C. Dawson - Stone & Webster Engineering Corporation
Watertown Arsenal Reactor by A. K. Sibley - Corps of Engineers

MONDAY AFTERNOON - OCTOBER 10, 1960

"Facilities for the Aircraft Nuclear Propulsion Program at the National Reactor Testing Station, Idaho" by W. E. Nims - Civilian Engineer Corps

"Impact of Nuclear Power on Shore Facilities of the Navy" by W. J. Christansen - Civilian Engineer Corps
"Construction of a Nuclear Power Plant in the Greenland Icecap" by J. T. Rhett and W. F. Reilly - Corps of Engineers

TUESDAY MORNING - OCTOBER 11, 1960

Power Division

Model and Prototype Research on Fish Ladders by B. M. Maclean and C. Cramer - Corps of Engineers
Fish Handling Facilities for Baker River Hydroelectric Project by W. W. Wayne - Stone & Webster Engineering Corporation
Scoping Lower Monumental Plant - by B. F. Smith, Corps of Engineers

TUESDAY AFTERNOON - OCTOBER 11, 1960

Economics of Cooling Ponds by C. W. Greene - Jackson & Moreland, Inc.
Niagara Power Project by G. R. Rich - Chas. T. Main, Inc.
Passamaquoddy Tidal Power Project by J. W. Leslie - Corps of Engineers

1961 Meetings

The next Spring meeting will be held at Phoenix, Arizona, April 10-14, 1961. The theme of the meeting is to be "Water Resources Development" and the Power Division expects to present several papers. The next annual meeting will be in New York October 16-20, 1961, and it is hoped that a symposium on Pumped Storage Projects will be presented. Several important pumped storage projects are now under construction or planned and the symposium would deal with economic aspects, system planning, and differences between European and American concepts.

International Commission on Large Dams

Members of the Power Division are reminded that the Seventh Congress of the International Commission on Large Dams will be held in Rome, June 1961. Papers will be presented on four questions:

Question No. 24: The selection, processing and specification of aggregates for concrete for large dams.
Question No. 25: Underground work in connection with large dams.
Question No. 26: Modern techniques of concrete dams for wide valleys and ancillary works.
Question No. 27: Sealing of earth and rockfill dams with bitumen and other materials.

Study tours of interesting dams will be available after the meeting.

Future Newsletters

News of interest from local sections or individual engineers will be welcomed for use in future Newsletters.

COMBINED INDEX TO ASCE PUBLICATIONS

For complete coverage of the Society's 1959 year in print, there is now a Combined Index covering the Division Journals, Transactions, and Civil Engineering. Also included are reprints of the Proceedings Abstracts that are published each month in Civil Engineering. The price of the Combined Index (ASCE publication 1960-10) is $2.00 with the usual 50% discount to members. The coupon herewith will make ordering easy:

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Newsletter Editor